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NATIONAL DAM INSPECTION PROGRAM. WADHAM CREEK DAM (NDI-PA-00547--ETC(U)
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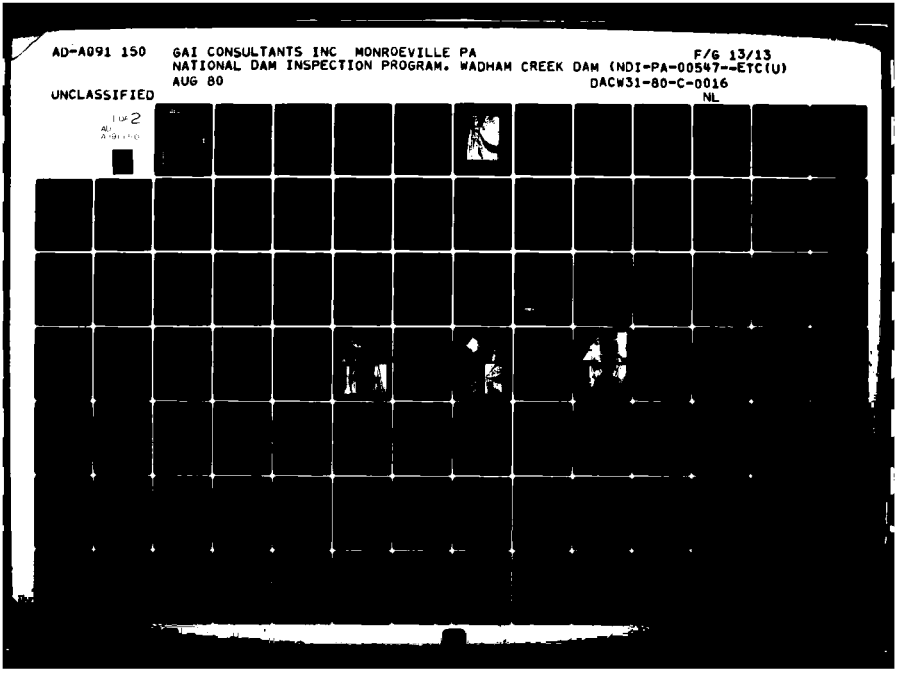

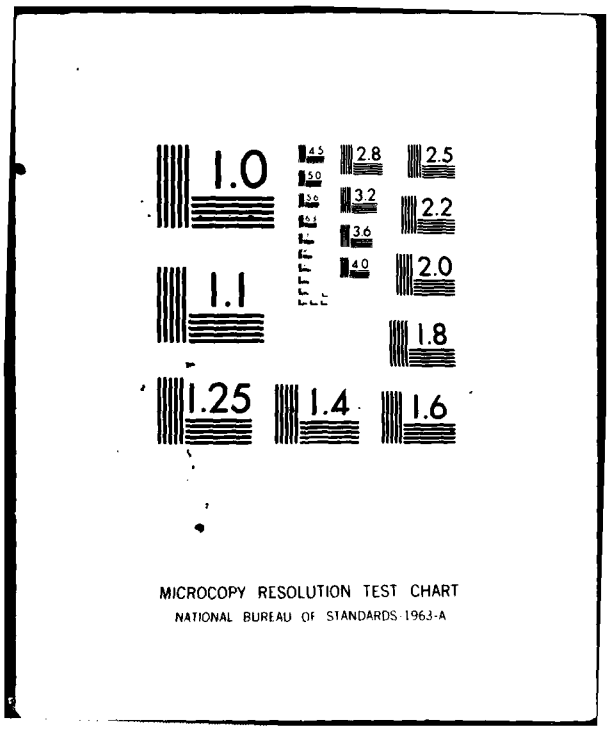
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SUSQUEHANNA RIVER BASIN
WADHAM CREEK, LUZERNE COUNTY

PENNSYLVANIA

WADHAM CREEK DAM

NDI I.D. No. PA-00547
PENNDER I.D. No. 40-209

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

PREPARED FOR

DEPARTMENT OF THE ARMY
Baltimore District, Corps of Engineers
Baltimore, Maryland 21203

DACW31-80-C-0016

PREPARED BY

GAI CONSULTANTS, INC.
570 BEATTY ROAD
MONROEVILLE, PENNSYLVANIA 15146

AUGUST 1980

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PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topograhic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through frequent inspections can unsafe conditions be detected and only through continued care and maintenance can these conditions be prevented or corrected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established guidelines, the spillway design flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. The spillway design flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition, and the downstream damage potential.

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

ABSTRACT

Wadham Creek Dam: NDI I.D. No. PA-00547

Owner:	Borough of Plymouth
State Located:	Pennsylvania (PennDER I.D. No. 40-209)
County Located:	Luzerne
Stream:	Wadham Creek
Inspection Date:	24 April 1980
Inspection Team:	GAI Consultants, Inc. 570 Beatty Road Monroeville, Pennsylvania 15146

Based on a visual inspection, operational history, and available engineering data, the dam is considered to be in fair condition.

The size classification of the facility is small and its hazard classification is considered to be high. In accordance with the recommended guidelines, the Spillway Design Flood (SDF) for the facility ranges between the 1/2-PMF (Probable Maximum Flood) and the PMF. Due to the high potential for damage to downstream structures and possible loss of life that could be associated with a sudden breach of the embankment, the SDF is considered to be the PMF. Results of hydrologic and hydraulic analysis indicate the spillway system for Wadham Creek Dam is capable of passing and/or storing the PMF. Consequently, the spillway is considered adequate.

It is recommended that the owner immediately:

a. Develop a formal warning system to notify downstream residents should hazardous conditions develop. Included in the plan should be provisions for around-the-clock surveillance of the facility during periods of unusually heavy precipitation.

b. Develop formal manuals of maintenance and operation for the facility and implement an effective program of routine maintenance.

c. Remove the trees and cyclone fence immediately downstream of the emergency spillway structure to provide an unobstructed flow path.

d. Restrict use of the embankment as an access route and/or take remedial measures (such as installing gravel walkways and fences) to limit wear and vandalism.

GAI Consultants, Inc.

Bernard M. Mihalcin

Bernard M. Mihalcin, P.E.

Approved by:

JAMES W. PECK
Colonel, Corps of Engineers
District Engineer

COMMONWEALTH OF PENNSYLVANIA
REGISTERED PROFESSIONAL
BERNARD M. MIHALCIN
ENGINEER
20371-E

Date 25 August 80

12 Sep 80

OVERVIEW PHOTOGRAPH

TABLE OF CONTENTS

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PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM
WADHAM CREEK DAM
(NDI-PA-00547, PENNDER-40-209)
Susquehanna River Basin, Wadham Creek, Luzerne County, Pennsylvania
Phase I Inspection Report.

SECTION 1
GENERAL INFORMATION

1.0 Authority.

The Dam Inspection Act, Public Law 92-367, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a program of inspection of dams throughout the United States.

1.1 Purpose.

The purpose is to determine if the dam constitutes a hazard to human life or property.

1.2 Description of Project.

a. Dam and Appurtenances. Wadham Creek Dam is an L-shaped, 30-foot high, earth and rockfill embankment approximately 400 feet long, including spillway. The facility is constructed with both service and emergency spillways. The service spillway is a 14-foot by 14-foot, uncontrolled, concrete, drop inlet type structure connected to a 7-foot by 7-foot, horizontal discharge conduit. The structure is located along the upstream slope near the center of the embankment. The service spillway is also equipped with a 24-inch diameter, cast iron, low flow outlet that discharges normal daily flows into the stream. No control is provided on the outlet. The emergency spillway consists of a 102-foot long, ogee shaped weir set in an uncontrolled, rectangular, concrete channel located near the left abutment. The dam is designed as a debris collection structure; has no permanent pool and, thus, no blowoff outlet.

b. Location. Wadham Creek Dam is located across Wadham Creek in Plymouth Borough, Luzerne County, Pennsylvania. The site is situated within a residential area less than 1/2-mile north of U.S. Route 11. The dam and watershed are contained within the Wilkes-Barre West and Kingston, Pennsylvania 7.5 minute U.S.G.S. topographic quadrangles (see Figure 1, Appendix E). The coordinates of the dam are N41° 14.5' and W75° 57.3'.

411 x VL

c.. Size Classification. Small (30 feet high, 12 acre-feet storage capacity at top of dam).

d. Hazard Classification. High (see Section 3.1.e).

e. Ownership. Borough of Plymouth
162 West Shawnee Avenue
Plymouth, Pennsylvania 18651
Attention: Mayor Edward Burns

f. Purpose. Debris collection.

g. Historical Data. Historical data from PennDER files are limited. The following history is primarily inferred from correspondence and the available construction drawings.

Wadham Creek Dam was designed by Bourquard, Geil & Associates of Harrisburg, Pennsylvania in 1958 and was constructed between the spring and fall of 1959. The purpose of the facility is to store rock and other debris from the upper watershed, thereby, preventing the blockage of pipes and culverts downstream of the dam. Brief progress reports indicate that construction inspection was provided by the Pennsylvania Department of Forests and Waters, Division of Flood Control.

Several undated photographs in PennDER files indicate the facility was constructed as originally designed. The visual inspection and several other design drawings (see Figures 8, 9, 10 and 11) indicate that the facility was modified in late 1960 or 1961. It appears that the modifications were required to provide a more effective low level outlet and impervious embankment barrier.

The facility has apparently functioned adequately since it was modified.

1.3 Pertinent Data.

a. Drainage Area (square miles). 1.0

b. Discharge at Dam Site.

Discharge Capacity of Outlet Conduit - Discharge curves are not available.

Discharge Capacity of Service Spillway at Maximum Pool ≅ 1400 cfs (see Appendix D, Sheet 13).

Discharge Capacity of Emergency Spillway at Maximum Pool ≈ 1610 cfs (see Appendix D, Sheet 16).

c. Elevation (feet above mean sea level). The following elevations were obtained through field measurements based on the elevation of the service spillway crest at 599.0 feet (see Appendix D, Sheet 1).

Top of Dam	606.0 (design). 605.5 (field).
Maximum Pool of Record	Not known.
Normal Pool	585.0
Service Spillway Crest	599.0
Emergency Spillway Crest	602.9
Upstream Inlet Invert	587.0
Downstream Outlet Invert	586.5
Service Spillway Outlet Invert	576.0 (design). 575.9 (field).
Streambed at Dam Centerline	578 (estimate).
Maximum Tailwater	Not known.

d. Reservoir Length (feet).

Top of Dam	500
Emergency Spillway Crest	450
Service Spillway Crest	400
Normal Pool	0

e. Storage (acre-feet).

Top of Dam	12
Emergency Spillway Crest	8
Service Spillway Crest	4
Normal Pool	0

f. Reservoir Surface (acres).

Top of Dam	1.4
Emergency Spillway Crest	1.2
Service Spillway Crest	0.8
Normal Pool	0

g. Dam.

Type	Earth and rock-fill.
Length	298 feet (excluding spillway).

	Height	30 feet (field measured; invert of service spillway outlet to embankment crest).
	Top Width	10 (design). 8 (field).
	Upstream Slope	2H:1V
	Downstream Slope	1.5H:1V
	Zoning	Rolled earth upstream, rockfill downstream. A two-stage coarse aggregate and sand filter separates the two zones (see Figure 5).
	Impervious Core	Upstream rolled earth zone is assumed to be impervious. Steel sheet piling also added as modification (see Figures 8 and 9).
	Cutoff	Steel sheet piling driven to rock.
	Grout Curtain	None indicated.
h.	Diversion Canal and Regulating Tunnels.	None.
i.	Service Spillway.	
	Type	14-foot by 14-foot, uncontrolled, concrete, drop

		inlet type structure connected to a 7-foot by 7-foot, horizontal discharge conduit.
	Crest Elevation	599.0 feet.
	Crest Length	42 feet.
j.	Emergency Spillway.	
	Type	Uncontrolled, rectangular, concrete channel with an ogee shaped weir.
	Crest Elevation	602.9 feet.
	Crest Length	102 feet.
k.	Outlet Conduit.	None (dry pool).
l.	Low Level Outlet.	24-inch diameter pipe connected to service spillway riser with inlet end protected by slotted concrete intake (see Figure 10).

SECTION 2
ENGINEERING DATA

2.1 Design.

a. Design Data Availability and Sources. No design reports, calculations or formal design data are available. Information contained in PennDER files is limited to detailed construction drawings and drawings for proposed modifications.

b. Design Features.

1. Embankment. Design features of the embankment are presented on Figures 3, 4, 5, 8, 9, and 11. As indicated, the embankment is a zoned earth and rockfill structure with a sheet pile cutoff and drainage provisions for rapid drawdown and seepage control. The upstream slope was set at 2H:1V whereas the downstream slope (comprised of rockfill) was set at 1-1/2H:1V. A two-stage filter system to prevent piping of fines is provided between the rockfill and rolled earth sections.

2. Appurtenant Structures.

a. Service Spillway. Design features of the service spillway are presented on Figures 2, 5, and 6. As indicated the spillway consists of a reinforced concrete drop inlet type structure that discharges into a reinforced concrete stilling basin at the downstream toe of the dam. A low flow outlet was provided, but, was modified after construction in accordance with Figures 9 and 10.

b. Emergency Spillway. Design features of the emergency spillway are presented on Figures 2 and 7. As indicated, the emergency spillway consists of a rectangular channel with an ogee shaped weir 102 feet long. Protection from undercutting of the downstream toe of the weir is provided by sheet piling driven to rock or a minimum depth of 10 feet (see Figure 7).

c. Specific Design Data and Criteria. No design data or information relative to design procedures are available other than the general notes contained on the construction drawings.

2.2 Construction Records.

No construction records are contained in available PennDER files or from the owner.

2.3 Operational Records.

No records of the day-to-day operation of the facility are available.

2.4 Other Investigations.

No formal investigations have been performed on this facility subsequent to its construction. Significant modifications were made to the structure in the early 1960's; however, aside from drawings contained in PennDER files, no other data is available.

2.5 Evaluation.

Engineering data are limited to drawings of the original design and modifications made subsequent to project completion. This data and the visual inspection are considered sufficient to make a reasonable Phase I evaluation of the facility.

SECTION 3
VISUAL INSPECTION

3.1 Observations.

a. General. The overall appearance of the facility suggests the dam and its appurtenances are in fair condition.

b. Embankment. Observations made during the visual inspection indicate the embankment is in fair condition. Minor sloughing was observed about midway along the upstream slope to the right of the service spillway (see Photographs 3 and 9). Many signs of misuse or abuse of the facility by the local populace were also in evidence. Access to the facility is not restricted and, consequently, the embankment has been thoroughly trampled and overridden with bicycles. This has resulted in several ruts and many unvegetated walking paths (see Photographs 1, 2, 3 and 5). Large areas where riprap has been displaced are commonplace. Normal routine maintenance appears insufficient.

c. Appurtenant Structures.

1. Service Spillway. The service spillway was observed to be in good condition. Some minor cracking was evident along the throat, but, is not considered significant. The outlet channel is cluttered with debris that should be removed regularly (see Photographs 2, 3, 4 and 10).

The intake structure for the low level outlet is also in need of maintenance. The top plate has been removed and debris is piling up around the intake slots (see Photograph 6).

2. Emergency Spillway. The condition of the emergency spillway is considered good. Minor shrinkage cracks around the joints and some spalling were observed.

The emergency spillway discharges into a large flat field (community baseball field) that slopes very gently downstream and along the riprapped embankment. A cyclone fence has been erected around the entire field. This fence, backed by large trees, stands about 20 feet downstream of the ogee weir and appears to present a potential for discharge obstruction (see Photographs 1, 7 and 8).

d. Reservoir Area. Wadham Creek Dam is situated within the community of Plymouth, Pennsylvania. The majority of the watershed has been strip mined; however, portions of the watershed contain steep, heavily forested slopes (see Photograph 1).

e. Dowstream Channel. The less than 1-mile reach between Wadham Creek Dam and the Susquehanna River is heavily populated with many residences located near the stream (see Photograph 11). It is estimated that significant damage could be incurred and more than a few lives lost along this reach as a result of an embankment breach. Consequently, the hazard classification is considered to be high.

3.2 Evaluation.

The overall condition of the facility is considered to be fair. The majority of the deficiencies noted by the inspection team can be attributed to the general misuse of the facility by the local populace and lack of a regular maintenance program. A formal maintenance program is recommended in addition to improvements that will protect the embankment from its daily use as an access route by the local populace.

SECTION 4
OPERATIONAL PROCEDURES

4.1 Normal Operating Procedure.

The facility is totally self-regulating. The dam is designed to retain water only as the result of a storm event. Normal daily stream flow is discharged via the low flow outlet through the service spillway discharge conduit and into the stream below. Flows in excess of the capacity of the outlet are stored until they can be safely discharged by either the outlet, service or emergency spillway. No mechanical operating devices are associated with this facility. A formal manual of operation and maintenance was reportedly developed and submitted to the owner by the PennDER, Bureau of Completed Projects; however, the owner was not able to produce the manual for review by the inspection team.

4.2 Maintenance of Dam.

No formal maintenance program is apparent at this facility. Maintenance is the responsibility of the Borough of Plymouth and is supposed to be performed in accordance with the procedures and guidelines contained in the formal Operations and Maintenance Manual developed by the PennDER.

4.3 Maintenance of Operating Facilities.

There are no mechanical operating devices associated with this facility.

4.4 Warning System.

No formal warning system is presently in effect. The Mayor of Plymouth stated that visual inspection and surveillance of the dam are initiated by a National Weather Service forecast of 2.5 inches of rain in a 3-hour period (flash flood watch). No evacuation plans are available. Emergency action is under the authority of the Mayor of Plymouth.

4.5 Evaluation.

No formal operations or maintenance manuals are currently available. Such manuals are recommended to ensure

the future proper care of the facility. The manuals should include a formal warning system that provides for around-the-clock surveillance of the facility during periods of unusually heavy rainfall and for the notification of downstream residents should hazardous conditions develop.

SECTION 5
HYDROLOGIC/HYDRAULIC EVALUATION

5.1 Design Data.

Information contained in PennDER files does not reveal the design procedures used to size the spillways. A design review in PennDER files indicated that the combined design discharge capacity of the spillways is about 3000 cfs which exceeds the required capacity of 950 cfs as defined by the Pennsylvania "C" Curve.

5.2 Experience Data.

Daily records of reservoir levels and/or spillway discharge are not available.

5.3 Visual Observations.

Two particular conditions observed by the field team are considered potentially hazardous to the facility since they present potential obstructions to the free maximum service and emergency spillway discharges. The potential to obstruct flow is magnified when the primary function of facility (debris dam) is considered. Specifically noted were the cyclone fence downstream of the emergency spillway (see Photograph 7) and the lack of maintenance which resulted in an accumulation of debris in the service spillway outlet (see Photograph 10). The cyclone fence, in particular, is backed by several large evergreen trees and situated between the dam and a relatively flat baseball field through which emergency flow would be directed. Under large flows debris which might lodge against the fence could increase backwater and effectively reduce the spillway discharge capacity which could then lead to embankment overtopping.

5.4 Method of Analysis

The facility has been analyzed in accordance with the recommended guidelines established by the U. S. Army, Corps of Engineers, Baltimore District, for Phase I hydrologic and hydraulic evaluations. The analysis has been performed utilizing a modified version of the HEC-1 program developed by the U. S. Army, Corps of Engineers, Hydrologic Engineering Center, Davis, California. Analytical capabilities of

the program are briefly outlined in the preface contained in Appendix D.

5.5 Summary of Analysis.

a. Spillway Design Flood (SDF). In accordance with procedures and guidelines contained in the National Guidelines for Safety Inspection of Dams for Phase I Investigations, the Spillway Design Flood (SDF) for Wadham Creek Dam ranges between the 1/2-PMF (Probable Maximum Flood) and the PMF. This classification is based on the relative size of the dam (small), and the potential hazard of dam failure to downstream developments (high). Due to the high potential for downstream damage and possible loss of life, the SDF for this facility is considered to be the PMF.

b. Results of Analysis. Wadham Creek Dam was evaluated under near normal design operating conditions. That is, the reservoir was initially drawn down to the zero storage elevation of approximately 585.0 feet. The low level outlet conduit was assumed to be non-functional for the purpose of analysis, since the flow capacity of this conduit is not such that it would significantly increase the total discharge capabilities of the facility. The service spillway consists of a 7-foot by 7-foot concrete conduit connected to a drop inlet structure, with a 14-foot by 14-foot square inlet. The emergency spillway consists of a concrete ogee-type weir structure which discharges into a wide and flat recreational area adjacent to the dam. The possibility of reservoir inflow from the diversion channel and culvert of the nearby Duffey Run watershed was also included as part of the analysis. All pertinent engineering calculations relative to the evaluation of Wadham Creek Dam are provided in Appendix D.

Overtopping analysis (using the Modified HEC-1 Computer Program) indicated that the discharge/storage capacity of Wadham Creek Dam can accommodate storms in excess of the PMF. The maximum total spillway capacity of 3010 cfs was found to be in excess of the peak PMF inflow of about 2740 cfs. The peak PMF inflow was not attenuated by the discharge/storage capabilities of the dam, as the resulting PMF peak outflow was also about 2740 cfs. Under the PMF event, the reservoir level rose as high as elevation 605.2, or about 0.3 feet below the low top of dam (Appendix D, Summary Input/Output Sheets, Sheets B and C).

Several points must be noted here, however. It was

assumed in the analysis that the service spillway would discharge freely under all flood conditions. It is possible that due to the nature of the dam and watershed, and the apparent lack of maintenance, the spillway culvert could clog to some extent with debris. It was also assumed that the emergency spillway would discharge freely. However, there is the possibility that the fence immediately downstream of the weir could accumulate debris, and consequently, the resulting tailwater levels could greatly reduce the discharge capabilities of the structure. Finally, it must be noted that the methods used to estimate reservoir inflow are primarily for natural watersheds. It was observed during the field inspection that the major watercourses within the drainage basin consisted of concrete lined channels, which ultimately could lead to peak inflows much higher than those computed here.

5.6 Spillway Adequacy.

Wadham Creek Dam was found to be capable of passing and/or storing its SDF (the PMF), and therefore, its spillway is considered to be adequate.

SECTION 6
EVALUATION OF STRUCTURAL INTEGRITY

6.1 Visual Observations.

a. Embankment. Based on visual observations, the embankment is considered to be in fair structural condition. The deficiencies noted are generally attributable to the misuse of the facility by the local populace and the lack of adequate maintenance provided by the owner. Remedial steps should be taken to correct these problems and prevent their reoccurrance.

b. Appurtenant Structures.

1. Service Spillway. The condition of the service spillway is considered to be good. Minor concrete deterioration was observed, but, is not considered to be significant at present. Special care should be taken to keep the spillway and outlet structures clear of potentially obstructing debris.

2. Emergency Spillway. The emergency spillway is considered to be in good condition. Large trees and a cyclone fence are situated within 20 feet of the weir that promote the potential for obstructing free flow and should be removed.

6.2 Design and Construction Techniques.

Other than design and modification drawings, no data are available that confirm methods of analysis, design or construction. Available drawings indicate, however, that the facility contains the required elements of accepted practice relative to modern earth and rockfill dam design. Local government officials stated that extensive deep mining has been performed in the area of the dam and reservoir. A cursory review of available mine maps provided by the officials revealed that undermining may in fact be a possibility in the general locale of the facility. Subsidence damage to the embankment coincident with spillway discharge appears remote; however, routine inspection of the facility to check for subsidence damage would appear prudent.

6.3 Past Performance.

Records indicate the facility has performed adequately since its construction.

6.4 Seismic Stability.

The dam is located within Seismic Zone No. 1 and may be subject to minor earthquake induced dynamic forces. As the facility appears well constructed and sufficiently stable, it is believed it can withstand the expected dynamic forces; however, no calculations and/or investigations were performed to confirm this opinion.

SECTION 7
ASSESSMENT AND RECOMMENDATIONS FOR REMEDIAL MEASURES

7.1 Dam Assessment.

a. Safety. The results of this evaluation indicate the facility is in fair condition with deficiencies primarily attributable to a general lack of routine maintenance.

The size classification of the facility is small and its hazard classification is high. In accordance with the recommended guidelines, the Spillway Design Flood (SDF) for the facility ranges from the 1/2-PMF (Probable Maximum Flood) to the PMF. Due to the high potential for downstream damage and loss of life that would be associated with a sudden failure of the embankment, the SDF for this facility is considered to be the PMF. Results of the hydrologic and hydraulic analysis indicate that the spillway system for Wadham Creek Dam is capable of passing and/or storing the PMF. Consequently, the spillway system is considered adequate.

b. Adequacy of Information. The available data are considered sufficient to make a reasonable Phase I assessment of the facility.

c. Urgency. The following recommendations should be implemented immediately.

d. Necessity for Additional Investigations. No additional studies are deemed necessary at this time.

7.2 Recommendations/Remedial Measures.

It is recommended that the owner immediately:

a. Develop a formal warning system to notify downstream residents should hazardous conditions develop. Included in the plan should be provisions for around-the-clock surveillance of the facility during periods of unusually heavy precipitation.

b. Develop formal manuals of maintenance and operation for the facility and implement an effective program of routine maintenance.

c. Remove the trees and cyclone fence immediately downstream of the emergency spillway structure to provide an unobstructed flow path.

d. Restrict use of the embankment as an access route and/or take remedial measures (such as installing gravel walkways and fences) to limit wear and vandalism.

APPENDIX A

VISUAL INSPECTION CHECKLIST AND FIELD SKETCH

CHECK LIST
VISUAL INSPECTION
PHASE 1

NAME OF DAM Wadham Creek Dam STATE Pennsylvania COUNTY Luzerne

NDI # PA — 00547 PENNDER # 40-209

TYPE OF DAM Earth-Rockfill SIZE Small HAZARD CATEGORY High

DATE(S) INSPECTION 24 April 1980 WEATHER Clear TEMPERATURE 70 @ 3:00 pm

POOL ELEVATION AT TIME OF INSPECTION No Pool M.S.L.

TAILWATER AT TIME OF INSPECTION - M.S.L.

INSPECTION PERSONNEL	OWNER REPRESENTATIVES	*OTHERS*
B. M. Mihalcin		
D. J. Spaeder		
D. L. Bonk		

RECORDED BY B. M. Mihalcin

EMBANKMENT

ITEM	OBSERVATIONS/REMARKS/RECOMMENDATIONS NDI# PA - 00547
SURFACE CRACKS	None observed.
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	None observed.
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	Minor sloughing observed below the berm on the upstream embankment face to the right of the service spillway. Unprotected crest is utilized as a foot and bicycle path and, thus, has no grass cover. Bicycle path also at bend in embankment on downstream face.
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST	Horizontal - Good. Vertical - Good (see "Profile of Dam Crest", Appendix A).
RIPRAP FAILURES	Portions of the riprap on both slopes have been displaced. This can be attributed to the fact that the facility is located within an urban area and is subject to casual vandalism and misuse by the local populace.
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	Good condition. Bicycle rut at right abutment-embankment junction. Gravel covered roadway and adjacent drainage ditch located at left abutment embankment junction.

EMBANKMENT

ITEM	OBSERVATIONS/REMARKS/RECOMMENDATIONS NDI# PA - 00547
DAMP AREAS IRREGULAR VEGETATION (LUSH OR DEAD PLANTS)	None observed - Dry pool.
ANY NOTICEABLE SEEPAGE	None observed - Dry pool.
STAFF GAGE AND RECORDER	None.
DRAINS	None observed functioning.

OUTLET WORKS

ITEM	OBSERVATIONS/REMARKS/RECOMMENDATIONS NDI# PA - 00547
INTAKE STRUCTURE	Low level inlet - concrete in good condition with evidence of minor spalling and cracking. Metal grate atop inlet is missing. Debris observed inside inlet about 3.5 feet from the top. Should be cleaned periodically.
OUTLET CONDUIT (CRACKING AND SPALLING OF CONCRETE SURFACES)	24-inch diameter pipe from low level inlet to throat of service spillway.
OUTLET STRUCTURE	Outlet conduit discharges into the service spillway.
OUTLET CHANNEL	See "Discharge Channel", Sheet 6 of 8.
GATE(S) AND OPERATIONAL EQUIPMENT	None.

EMERGENCY SPILLWAY

ITEM	OBSERVATIONS/REMARKS/RECOMMENDATIONS NDI# PA - 00547
TYPE AND CONDITION	Uncontrolled, rectangular, concrete channel with an ogee shaped weir. Good condition. Minor shrinkage cracks observed throughout along with spalling around the joints.
APPROACH CHANNEL	None.
SPILLWAY CHANNEL AND SIDEWALLS	No defined channel. Spillway discharges into a baseball field. Cyclone fence located immediately downstream (≈ 20 feet from the crest). Concrete sidewalls in good condition. Fence may collect debris and retard spillway discharges. Should be removed along with the trees behind it (see Photos 1, 2, 7, 8).
STILLING BASIN PLUNGE POOL	N/A.
DISCHARGE CHANNEL	Open flat field.
BRIDGE AND PIERS EMERGENCY GATES	None.

SERVICE SPILLWAY

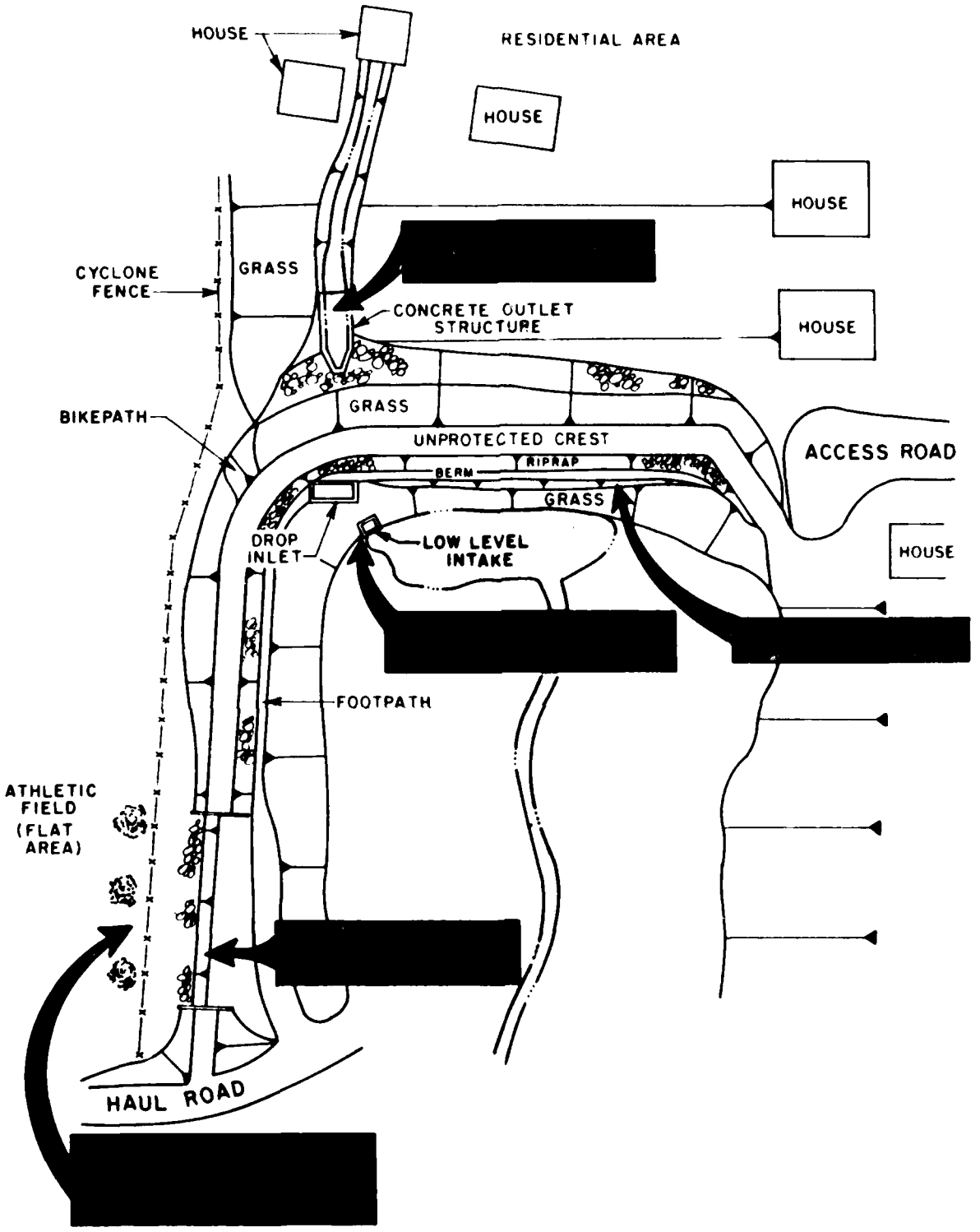
ITEM	OBSERVATIONS/REMARKS/RECOMMENDATIONS NDI# PA - 00547
TYPE AND CONDITION	Concrete drop inlet with rectangular shaped orifice. Concrete in good condition. Some cracking evident near channel throat - not significant.
APPROACH CHANNEL	None.
OUTLET STRUCTURE	7-foot by 7-foot box culvert in good condition. Debris should be removed from throat area and stilling basin.
DISCHARGE CHANNEL	Rock lined channel with bedrock floor immediately beyond the outlet structure. Channel constricted by another culvert that runs beneath a dwelling approximately 300 feet downstream.

INSTRUMENTATION

ITEM	OBSERVATIONS/REMARKS/RECOMMENDATIONS NDI# PA - 00547
MONUMENTATION SURVEYS	None.
OBSERVATION WELLS	None.
WEIRS	None.
PIEZOMETERS	None.
OTHERS	

RESERVOIR AREA AND DOWNSTREAM CHANNEL

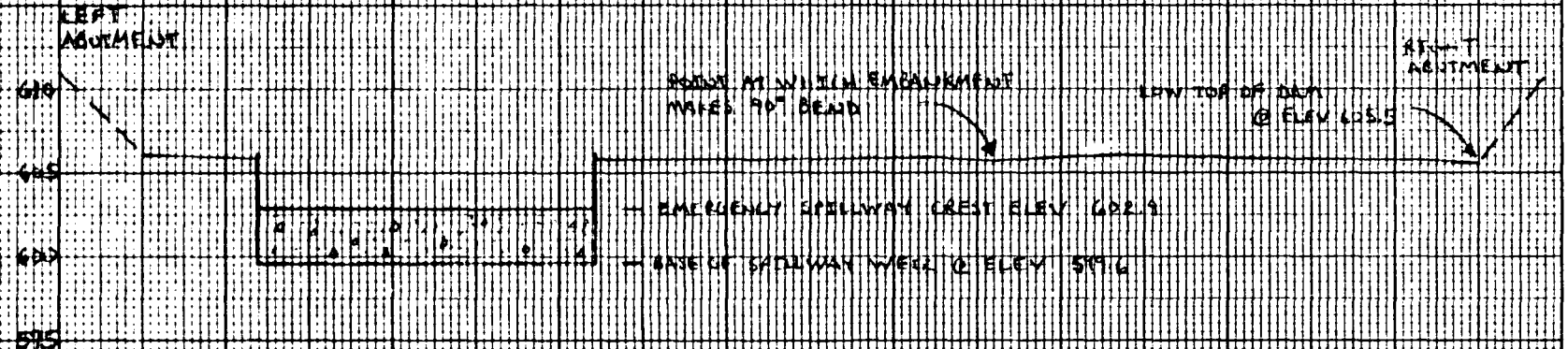
ITEM	OBSERVATIONS/REMARKS/RECOMMENDATIONS NDI# PA - 00547
SLOPES: RESERVOIR	Brush covered strip mine area with steep slopes.
SEDIMENTATION	Debris Dam - about 3 to 5 feet of sediment observed around low level intake structure.
DOWNSTREAM CHANNEL (OBSTRUCTIONS, DEBRIS, ETC.)	The stream flows through an urban area prior to discharging into the Susquehanna River. Along its path it is directed beneath several roads and culverts which could potentially become obstructed. The purpose of the facility is to reduce the potential for this type of obstruction.
SLOPES: CHANNEL VALLEY	Unlined, trapezoidal shaped channel through an urban area.
APPROXIMATE NUMBER OF HOMES AND POPULATION	The less than 1-mile reach between Wadham Creek Dam and the Susquehanna River is heavily populated with many residences located near the stream. It is estimated that more than a few lives would be lost as the result of an embankment breach. The hazard classification is considered to be high.



WADHAM CREEK DAM
GENERAL PLAN - FIELD INSPECTION NOTES

WADHAM CREEK DAM

PROFILE OF DAM CREST
FROM FIELD SURVEY



SCALE: VERTICAL 1 IN = 10 FT
HORIZONTAL 1 IN = 50 FT

APPENDIX B

ENGINEERING DATA CHECKLIST

CHECK LIST
ENGINEERING DATA
PHASE I

NAME OF DAM Wadham Creek Dam

ITEM	REMARKS NDI# PA - 00547
PERSONS INTERVIEWED AND TITLE	Edward Burns, Mayor, Borough of Plymouth. Steven Senko, Assistant Borough Secretary, Borough of Plymouth (via telephone).
REGIONAL VICINITY MAP	See Figure 1, Appendix E.
CONSTRUCTION HISTORY	Limited Correspondence relative to the construction of Wadham Creek Dam are available in PennDER files, see Section 1.2.g of report.
AVAILABLE DRAWINGS	Complete set of design drawings, dated 1958, and drawings of modifications, dated 1960, by Bourquard, Geil and Associates of Harrisburg, Pennsylvania, available from PennDER, Bureau of Design. None available from owner.
TYPICAL DAM SECTIONS	See Figures 2, 4, and 5, Appendix E. (design). See Figures 8, 9, and 11, Appendix E. (modifications).
OUTLETS: PLAN DETAILS DISCHARGE RATINGS	See Figures 9 and 10, Appendix E. Discharge rating curves are not available.

CHECK LIST
ENGINEERING DATA
PHASE I
(CONTINUED)

ITEM	REMARKS NDI# PA - 00547
SPILLWAY: PLAN SECTION DETAILS	See Figures 2, 5, and 6, Appendix E. (Service Spillway). See Figures 2 and 7, Appendix E. (Emergency Spillway).
OPERATING EQUIP- MENT PLANS AND DETAILS	N/A.
DESIGN REPORTS	None Available.
GEOLOGY REPORTS	None Available.
DESIGN COMPUTATIONS: HYDROLOGY AND HYDRAULICS STABILITY ANALYSES SEEPAGE ANALYSES	None Available.
MATERIAL INVESTIGATIONS: BORING RECORDS LABORATORY TESTING FIELD TESTING	See Figure 3, Appendix E.

CHECK LIST
ENGINEERING DATA
PHASE I
(CONTINUED)

ITEM	REMARKS NDI# PA - 00547
BORROW SOURCES	Unknown, but probably from within vicinity of borrow areas for post-construction modifications depicted on Figure 8, Appendix E.
POST CONSTRUCTION DAM SURVEYS	See Figures 8, 9, 10 and 11, Appendix E.
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	None Available.
HIGH POOL RECORDS	Local cloud burst in 1966 reportedly caused reservoir to rise to within two feet of the top of the embankment.
MONITORING SYSTEMS	None.
MODIFICATIONS	See Figures 8, 9, 10 and 11, Appendix E. Sediment removed from reservoir by PennDER in 1966, 1972, and 1975.

CHECK LIST
ENGINEERING DATA
PHASE I
(CONTINUED)

ITEM	REMARKS NDI# PA - 00547
PRIOR ACCIDENTS OR FAILURES	Flood of June 1972 caused erosion damage to residential foundations and masonry stream channel sidewalls. Dam did not overtop.
MAINTENANCE: RECORDS MANUAL	None Available.
OPERATION: RECORDS MANUAL	None Available.
OPERATIONAL PROCEDURES	Ficility is self-regulating.
WARNING SYSTEM AND/OR COMMUNICATION FACILITIES	No formal system. Visual inspection and surveillance of Wadham Creek Dam are initiated by a forecast of 2.5 inches of rainfall in a 3-hour period. Inspector at dam has radio communication with other borough personnel. Evacuation plans are not available. The mayor of the borough authorizes all actions at the dam.
MISCELLANEOUS	Upstream reservoir area has been extensively strip mined and concrete channels have been installed to transport runoff to reservoir. Dam and reservoir are reportedly undermined (owner has subsurface maps showing areas where support columns may have been "robbed").

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CHECK LIST
HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA

NDI ID # PA-00547
PENNDER ID # 40-209

SIZE OF DRAINAGE AREA: 1.0 square miles

ELEVATION TOP NORMAL POOL: 585.0 STORAGE CAPACITY: 0 (no pool).

ELEVATION TOP FLOOD CONTROL POOL: - STORAGE CAPACITY: -

ELEVATION MAXIMUM DESIGN POOL: - STORAGE CAPACITY: -

ELEVATION TOP DAM: 605.5 STORAGE CAPACITY: 11.8 acre-feet.

SPILLWAY DATA

CREST ELEVATION: 599.0 (service); 602.9 (emergency).

TYPE: Drop inlet (service); rectangular channel with ogee (emergency).

CREST LENGTH: 14' by 14' opening (service); 102' (emergency).

CHANNEL LENGTH: N/A.

SPILLOVER LOCATION: Embankment center (service); left abutment (emergency).

NUMBER AND TYPE OF GATES: None.

OUTLET WORKS

TYPE: Uncontrolled low flow outlet.

LOCATION: Base of service spillway.

ENTRANCE INVERTS: 587.0 feet.

EXIT INVERTS: 586.5 feet.

EMERGENCY DRAWDOWN FACILITIES: None.

HYDROMETEOROLOGICAL GAGES

TYPE: None.

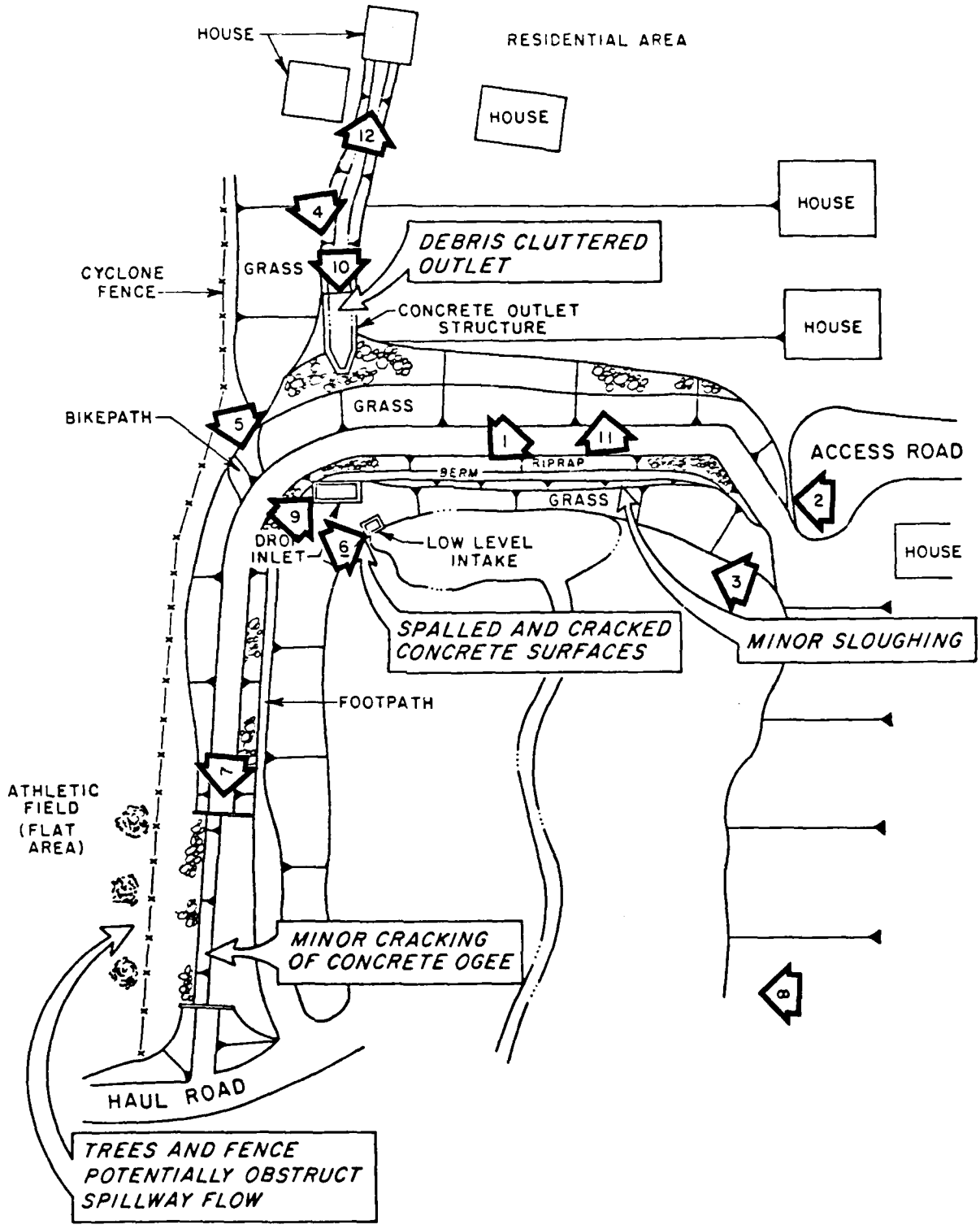
LOCATION: -

RECORDS: -

MAXIMUM NON-DAMAGING DISCHARGE: Not known.

APPENDIX C

PHOTOGRAPHS



WADHAM CREEK DAM
PHOTOGRAPH KEY MAP

PHOTOGRAPH 1	View of the area behind the embankment looking upstream from the embankment crest.
PHOTOGRAPH 2	View across the embankment crest as seen from the right abutment.
PHOTOGRAPH 3	View of the upstream embankment face between the service spillway and right abutment.
PHOTOGRAPH 4	View, looking upstream, of the service spillway outlet structure located at the downstream embankment toe.

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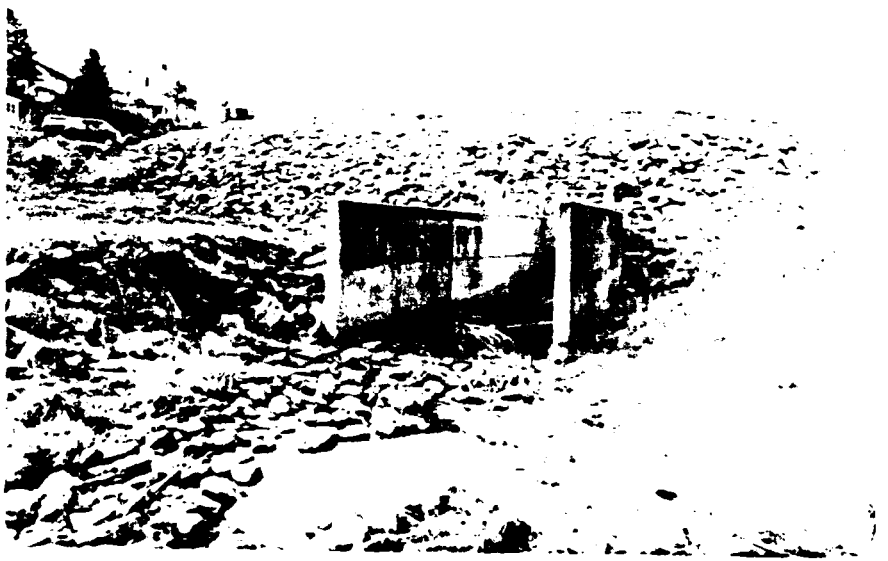


4

PHOTOGRAPH 5 — View of a large bare area on the downstream embankment face caused by local bicyclists.

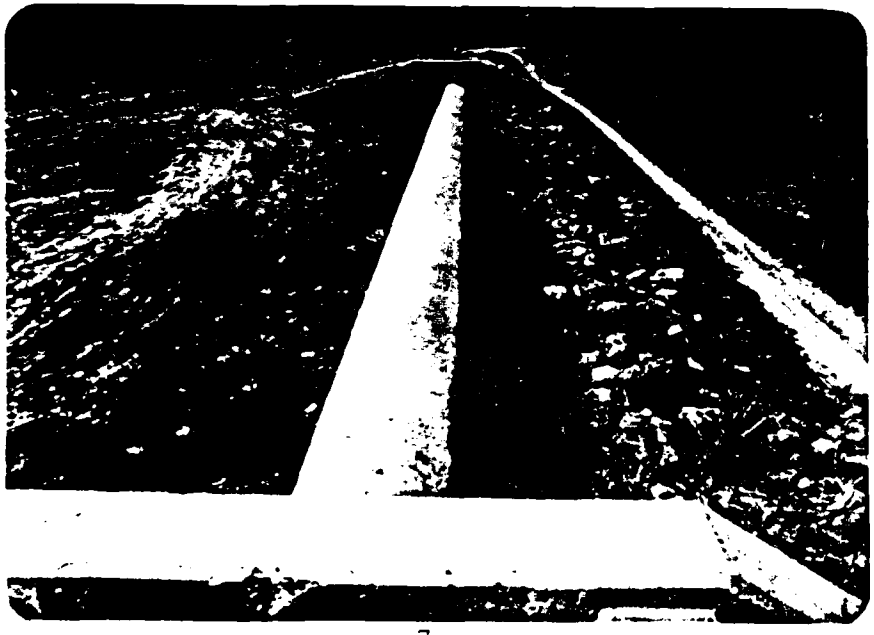
PHOTOGRAPH 6 — View of the inlet for the low flow outlet conduit located immediately below the service spillway riser.

PHOTOGRAPH 7 — View of the emergency spillway located at the left abutment.

PHOTOGRAPH 8 — View, looking downstream, of the emergency spillway as viewed from the right abutment hillside.

5

6



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8

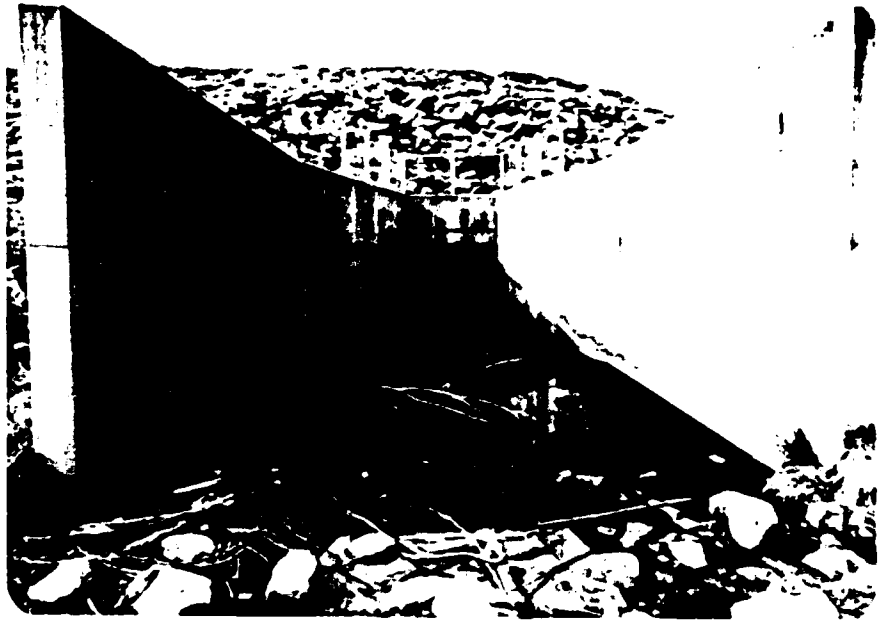
PHOTOGRAPH 9 View of the service spillway riser and low flow inlet.

PHOTOGRAPH 10 View of the debris cluttered service spillway discharge structure.

PHOTOGRAPH 11 View of the area immediately downstream of the embankment as seen from the embankment crest.

PHOTOGRAPH 12 View of a home constructed across the stream located about 300 feet downstream of the embankment.

9



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11

12

APPENDIX D

HYDROLOGY AND HYDRAULICS ANALYSES

PREFACE

The modified HEC-1 program is capable of performing two basic types of hydrologic analyses: 1) the evaluation of the overtopping potential of the dam; and 2) the estimation of the downstream hydrologic-hydraulic consequences resulting from assumed structural failures of the dam. Briefly, the computational procedures typically used in the dam overtopping analysis are as follows:

a. Development of an inflow hydrograph(s) to the reservoir.

b. Routing of the inflow hydrograph(s) through the reservoir to determine if the event(s) analyzed would overtop the dam.

c. Routing of the outflow hydrograph(s) from the reservoir to desired downstream locations. The results provide the peak discharge(s), time(s) of the peak discharge(s), and the maximum stage(s) of each routed hydrograph at the downstream end of each reach.

The evaluation of the hydrologic-hydraulic consequences resulting from an assumed structural failure (breach) of the dam is typically performed as shown below.

a. Development of an inflow hydrograph(s) to the reservoir.

b. Routing of the inflow hydrograph(s) through the reservoir.

c. Development of a failure hydrograph(s) based on specified breach criteria and normal reservoir outflow.

d. Routing of the failure hydrograph(s) to desired downstream locations. The results provide estimates of the peak discharge(s), time(s) to peak and maximum water surface elevations of failure hydrographs for each location.

HYDROLOGY AND HYDRAULIC ANALYSIS
DATA BASE

NAME OF DAM: WADHAM CREEK DAM

PROBABLE MAXIMUM PRECIPITATION (PMP) = 22.2 INCHES/24 HOURS (1)

STATION	1	2	3
STATION DESCRIPTION	WADHAM CREEK DAM		
DRAINAGE AREA (SQUARE MILES)	1.0		
CUMULATIVE DRAINAGE AREA (SQUARE MILES)	-		
ADJUSTMENT OF PMF FOR DRAINAGE AREA LOCATION (%) (1)			
6 HOURS	117.5		
12 HOURS	127.0		
24 HOURS	136.0		
48 HOURS	142.5		
72 HOURS	145.0		
SNYDER HYDROGRAPH PARAMETERS			
ZONE (2)	12		
C_p (3)	0.30		
C_t (3)	0.95		
L (MILES) (4)	1.5		
L_{ca} (MILES) (4)	0.7		
$t_p = C_t (L \cdot L_{ca})^{0.3}$ (HOURS)	0.96		
SPILLWAY DATA			
CREST LENGTH (FEET)	102		
FREEBOARD (FEET)	2.6		

(1) HYDROMETEOROLOGICAL REPORT 40, U.S. WEATHER BUREAU, 1965.

(2) HYDROLOGIC ZONE DEFINED BY CORPS OF ENGINEERS, BALTIMORE DISTRICT, FOR DETERMINATION OF SNYDER COEFFICIENTS (C_p AND C_t).

(3) SNYDER COEFFICIENTS

(4) L = LENGTH OF LONGEST WATERCOURSE FROM DAM TO BASIN DIVIDE.
L_{ca} = LENGTH OF LONGEST WATERCOURSE FROM DAM TO POINT OPPOSITE BASIN CENTROID

SUBJECT DAM SAFETY INSPECTION
WADHAM SETTLE DAM

BY JDS DATE 7-15-80 PROJ. NO. 79-303-547

CHKD. BY WJV DATE 8-4-80 SHEET NO. 1 OF 22

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DAM STATISTICS

HEIGHT OF DAM = 30 FT (FIELD MEASURED: INVERT OF SERVICE SPILLWAY OUTLET TO LOW TOP OF DAM)

NORMAL POOL STORAGE CAPACITY = 0 AC-FT (DEBRIS DAM)

MAXIMUM POOL STORAGE CAPACITY = 11.8 AC-FT (SHEET 3)
(@ LOW TOP OF DAM)

DRAINAGE AREA = 1.0 SQUARE MILE (PLANIMETERED ON USGS 7½ QUADS, WILKES-BARRE WEST AND KINGSTON, PA)

ELEVATIONS:

TOP OF DAM (DESIGN):	606.0	(FIG. 5)
TOP OF DAM (FIELD):	605.5	
CREST OF EMERGENCY SPILLWAY:	602.9	(FIELD SURVEY)
CREST OF SERVICE SPILLWAY RISER:	597.0	(FIG 5)
UPSTREAM LOW LEVEL INLET INVERT:	587.0	(FIG. 10)
LOW LEVEL OUTLET INVERT (INTO SERVICE SPILLWAY RISER):	586.5	(FIG. 10)
DOWNSTREAM SERVICE SPILLWAY OUTLET INVERT (DESIGN):	576.0	(FIG. 5)
DOWNSTREAM SERVICE SPILLWAY OUTLET INVERT (FIELD):	575.9	
STREAMBED AT DAM CENTERLINE:	≈ 578	(FIG 4, ESTIMATE)

JBJECT DAM SAFETY INSPECTION
WADHAM CREEK DAM

BY DDS DATE 7-29-80 PROJ. NO. 79-303-547

CHKD. BY WJV DATE 8-4-80 SHEET NO. 2 OF 20

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DAM CLASSIFICATION

DAM SIZE: SMALL (REF 1, TABLE 1)

HAZARD CLASSIFICATION: HIGH (FIELD OBSERVATION)

REQUIRED SDF: ½ PMF TO PMF (REF 1, TABLE 3)

HYDROGRAPH PARAMETERS

LENGTH OF LONGEST WATERCOURSE: $L = 1.5$ MI.

LENGTH OF LONGEST WATERCOURSE FROM DAM TO A POINT OPPOSITE BASIN CENTROID: $L_{CA} = 0.7$ MI.

(MEASURED ON USGS TOPO QUADS; KINGSTON AND WILKES-BARRE WEST, PA)

$C_t = 0.95$
$C_p = 0.30$

(SUPPLIED BY C.O.E.; ZONE 12, NORTH BRANCH SUSQUEHANNA RIVER BASIN)

SNYDER'S STANDARD LAG: $t_p = C_t (L \cdot L_{CA})^{0.3}$

$= 0.95 (1.5 \times 0.7)^{0.3}$

$= 0.96$ HOURS

NOTE: HYDROGRAPH VARIABLES USED HERE ARE DEFINED IN REF 2, IN SECTION ENTITLED "SNYDER SYNTHETIC UNIT HYDROGRAPH."

SUBJECT DAM SAFETY INSPECTION
WADHAM CREEK DAM

BY DJJ DATE 2-14-80 PROJ. NO. 79-203-547

CHKD. BY WJV DATE 2-4-80 SHEET NO. 3 OF 20

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RESERVOIR CAPACITY

IT IS ASSUMED THAT THE MODIFIED PRISMOIDAL RELATIONSHIP ADEQUATELY MODELS THE RESERVOIR SURFACE AREA - STORAGE RELATIONSHIP :

$$\Delta V_{1-2} = \frac{h}{3}\left(A_1 + A_2 + \sqrt{A_1 \cdot A_2}\right) \qquad \text{(REF 14, p. 15)}$$

WHERE ΔV_{1-2} = INCREMENTAL VOLUME BETWEEN ELEVATIONS 1 & 2, IN AC-FT,
h = ELEVATION 1 - ELEVATION 2, IN FT,
A_1 = SURFACE AREA AT ELEVATION 1, IN ACRES,
A_2 = SURFACE AREA AT ELEVATION 2, IN ACRES.

ELEVATION STORAGE TABLE:

RESERVOIR ELEVATION (FT)	A ① (AC)	ΔV_{1-2} (AC-FT)	TOTAL ② VOLUME (AC-FT)
585.0	0	—	0
588.0	0.07	0.07	0.1
590.0	0.13	0.20	0.3
592.0	0.24	0.36	0.6
594.0	0.39	0.62	1.3
596.0	0.57	0.95	2.2
598.0	0.75	1.32	3.5
(CREST OF RISER) 599.0	0.86	0.80	4.3
600.0	0.96	0.91	5.2
602.0	1.12	2.08	7.3
(CREST OF EMERGENCY SPILLWAY) 602.9	1.20	1.04	8.4
604.0	1.30	1.37	9.7
(LOW TOP OF DAM) 605.5	1.43	2.35	11.8
606.0	1.47	0.72	12.5
608.0	1.59	3.06	15.6
610.0	1.70	3.29	18.8
612.0	1.85	3.55	22.4

① PLANIMETERED ON FIG. 2; VALUES AT 599.0, 602.9, 605.5 BY LINEAR INTERPOLATION.
② $\Sigma\Delta V$, ROUNDED TO NEAREST 0.1 AC-FT.

SUBJECT DAM SAFETY INSPECTION
WADHAM CREEK DAM

BY DJS DATE 7-16-80 PROJ. NO. 79-203-547

CHKD. BY WJV DATE 2-4-81 SHEET NO. 4 OF 20

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PMP CALCULATIONS

- FROM REF. 9, FIG. 2, OBTAIN PMP VALUE FOR A BASIN OF DRAINAGE AREA 200 SQ. MI., 24-HOUR DURATION:

$P = 22.2$ INCHES

- FROM REF 9, FIG. 1, GEOGRAPHIC ADJUSTMENT FACTOR = 98%
(N 41°-14.5' x W 75°-57.3')

∴ CORRECTED RAINFALL INDEX = 21.8 INCHES

- RAINFALL DISTRIBUTION (REF. 9):

DURATION (HRS):	6	12	24	48	72
PERCENT OF INDEX RAINFALL:	117.5	127.0	136.0	142.5	145.0

- HOP BROOK FACTOR (ADJUSTMENT FOR BASIN SHAPE AND FOR THE LESSER LIKELIHOOD OF A SEVERE STORM CENTERING OVER A SMALL BASIN) FOR A DRAINAGE AREA OF 1.0 SQUARE MILE IS 0.80.
(REF 4, P. 48)

SUBJECT DAM SAFETY INSPECTION
WADHAM CREEK DAM

BY DJS DATE 7-16-80 PROJ. NO. 79-203-547

CHKD. BY WJ DATE 8-4-80 SHEET NO. 5 OF 20

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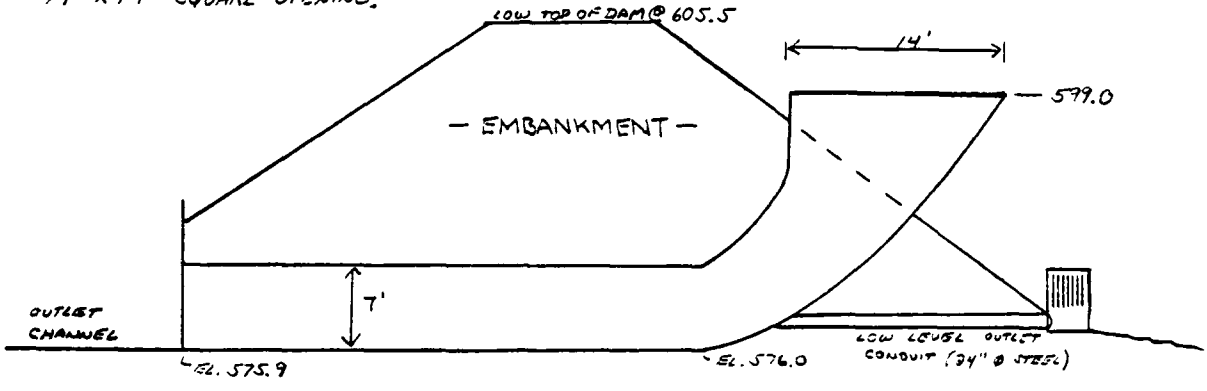
CAPACITY OF PRINCIPAL SPILLWAY

THE PRINCIPAL SPILLWAY CONSISTS OF A 7'x7' RECTANGULAR CONCRETE CONDUIT CONNECTED TO A DROP INLET STRUCTURE, WITH A 14' x 14' SQUARE OPENING.



(NOT TO SCALE)
(BASED ON FIELD SURVEY, FIG. 5, & FIG. 10)

DISCHARGE THROUGH THE CONDUIT WILL BE CALCULATED UNDER INLET CONTROL AND OUTLET CONTROL (IN ADDITION TO WEIR CONTROL AT THE INLET FOR LOW HEADS) IN ORDER TO DETERMINE WHICH CONDITION DICTATES.

I) <u>WEIR FLOW</u>: AT LOW HEADS, THE OPENING OF THE DROP INLET STRUCTURE WILL BEHAVE AS A SHARP-CRESTED WEIR:

$$Q = CLH^{3/2} \qquad \text{(REF 5, p. 5-3)}$$

WHERE Q = DISCHARGE, IN CFS,
C = COEFFICIENT OF DISCHARGE,
L = WEIR LENGTH, IN FT,
H = HEAD, IN FT.

BJECT DAM SAFETY INSPECTION
WADHAM CREEK DAM

BY JJS DATE 7-17-80 PROJ. NO. 79-203-547

CHKD. BY WJV DATE 8-4-80 SHEET NO. 6 OF 30

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WEIR FLOW RATING CURVE:

RESERVOIR ELEVATION (FT)	H (FT)	L ① (FT)	Q ② (CFS)
599.0	0	—	0
599.5	0.5	41	50
600.0	1.0	40	130
600.5	1.5	39	240
601.0	2.0	38	360
601.5	2.5	37	490
602.0	3.0	36	620
602.5	3.5	35	760
603.0	4.0	34	910
603.5	4.5	33	1050
604.0	5.0	32	1190
604.5	5.5	31	1330
605.0	6.0	30	1470

① L = WEIR LENGTH; BELOW ELEV. 606.0, THREE OF THE FOUR SIDES WILL CONTRIBUTE TO THE TOTAL WEIR LENGTH (SEE FIG. 5). SINCE THERE WILL BE FLOW OVER THE CORNERS, AN INCREASING PORTION OF THE WEIR LENGTH WILL BE INEFFECTIVE AS THE HEAD INCREASES. IT WILL BE ASSUMED THAT AT EACH OF THE TWO CORNERS FARTHEST FROM THE EMBANKMENT, THE PORTION OF THE LENGTH WHICH IS INEFFECTIVE WILL BE EQUAL TO THE HEAD. THUS, $L = 3(14) - 2H$

② $Q = CLH^{3/2}$, $C = 3.33$ (REF 5, p. 5-7)

SUBJECT DAM SAFETY INSPECTION
WADHAM CREEK DAM

BY DDS DATE 7-17-80 PROJ. NO. 79-203-547

CHKD. BY WJV DATE 8-11-80 SHEET NO. 7 OF 20

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II) DISCHARGE UNDER INLET CONTROL:

DUE TO THE GEOMETRY OF THE DROP INLET STRUCTURE, THE POINT OF CONTROL FOR DISCHARGE UNDER INLET CONTROL CONDITIONS IS ASSUMED TO BE AT THE ENTRANCE OF THE HORIZONTAL CONDUIT. DISCHARGE CAN BE ESTIMATED BY THE RELATIONSHIP

$$Q = C_n B D \sqrt{2g(H - C_n D)} \qquad \text{(SEE NOTE 1)}$$

WHERE Q = CULVERT DISCHARGE, IN CFS,
C_n = CONTRACTION COEFFICIENT = 0.8 ("ROUNDED" INLET),
B = WIDTH OF CULVERT = 7 FT,
D = HEIGHT OF CULVERT = 7 FT,
g = GRAVITATIONAL ACCELERATION CONSTANT = 32.2 FT/SEC^2,
H = HEAD, IN FT, MEASURED FROM INVERT OF CULVERT.

ALSO, DUE TO THE SHAPE OF THE CONDUIT, A CORRECTION FACTOR MUST BE INCLUDED TO ACCOUNT FOR THE ACTUAL CROSS-SECTION AREA:

$$A_{CONDUIT} = (3.8)(7) + (2)(1.6)\left(\frac{7+5.5}{2}\right) \qquad \text{(SEE FIG. 5)}$$
$$= 46.6 \text{ FT}^2$$

$$\therefore \quad Q = C_n (46.6)\sqrt{2g(H - C_n D)}$$
$$= (0.8)(46.6)\sqrt{64.4\,[H - (0.8)(7)]}$$
$$= 299.2\sqrt{H - 5.6}$$

THE INLET CONTROL RATING CURVE IS GIVEN ON SHEET 8.

NOTE 1: FROM OPEN CHANNEL FLOW, F.M. HENDERSON, MAC MILLAN PUBLISHING CO., INC., NEW YORK, 1966, PP. 263-264.

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INLET CONTROL RATING CURVE:

RESERVOIR ELEVATION (FT)	H ① (FT)	Q ② (CFS)	RESERVOIR ELEVATION (FT)	H ① (FT)	Q ② (CFS)
599.0	–	0	607.0	31.0	1510
600.0	24.0	1280	607.5	31.5	1520
602.0	26.0	1350	608.0	32.0	1540
604.0	28.0	1420	608.5	32.5	1550
604.5	28.5	1430	609.0	33.0	1570
605.0	29.0	1450	609.5	33.5	1580
605.5	29.5	1460	610.0	34.0	1590
606.0	30.0	1480			
606.5	30.5	1490			

① H = RESERVOIR ELEVATION − 576.0

② $Q = 299.2\sqrt{H-5.6}$

III) DISCHARGE UNDER OUTLET CONTROL:

DISCHARGE UNDER OUTLET CONTROL CONDITIONS CAN BE ESTIMATED BY BALANCING THE ENERGY EQUATION BETWEEN A POINT ON THE RESERVOIR WATER SURFACE AND A POINT ON THE TAILWATER SURFACE JUST DOWNSTREAM OF THE OUTLET.

$$\frac{P_R}{\gamma} + \frac{V_R^2}{2g} + Z_R = \frac{P_{TW}}{\gamma} + \frac{V_{TW}^2}{2g} + Z_{TW} + H_L \qquad \text{(REF 13, p. 66)}$$

WHERE $\frac{P_R}{\gamma} = \frac{P_{TW}}{\gamma}$ = ATMOSPHERIC PRESSURE HEAD @ THE RESPECTIVE WATER SURFACES = 0 FT,

$V_R^2/2g$ = VELOCITY HEAD AT RESERVOIR = 0 FT,

Z_R = RESERVOIR ELEVATION, IN FT,

$V_{TW}^2/2g$ = VELOCITY HEAD OF TAILWATER @ CULVERT OUTLET = VELOCITY HEAD IN CULVERT JUST UPSTREAM OF OUTLET, IN FT,

Z_{TW} = ELEVATION OF TAILWATER @ CULVERT OUTLET, IN FT,

UBJECT DAM SAFETY INSPECTION
WADHAM CREEK DAM

BY DTS DATE 7-21-80 PROJ. NO. 79-203-547

CHKD. BY WJV DATE 8-4-80 SHEET NO. 9 OF 20

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H_L = TOTAL HEAD LOSS, IN FT; CONSISTS OF ENTRANCE LOSS AT INLET, TRANSITION LOSS OR LOSS AT BEND, AND FRICTION LOSS.

$$\therefore \quad Z_R = Z_{TW} + \frac{V_{TW}^2}{2g} + H_L .$$

ESTIMATE H_L:

A) ENTRANCE LOSS: $H_e = 0.5 \frac{V_{ENTRANCE}^2}{2g}$ (REF 19, p. 5-49)

$$V_{ENTRANCE} = \frac{A_{CULVERT}}{A_{ENTRANCE}} V_{CULVERT}$$
$$= \frac{46.6}{(14)(14)} V_{CULVERT}$$

ASSUMING THAT THE TAILWATER VELOCITY AT THE OUTLET IS APPROXIMATELY EQUAL TO THE VELOCITY IN THE CULVERT ($V_{TW} = V_{CULVERT}$),

$$V_{ENTRANCE} = \frac{46.6}{(14)(14)} V_{TW}$$
$$= 0.24\, V_{TW}$$

$$H_e = (0.5) \frac{(0.24\, V_{TW})^2}{2g}$$
$$= 0.03 \frac{V_{TW}^2}{2g}$$

B) LOSS AT BEND: $H_B = 0.5 \frac{V_{BEND}^2}{2g}$ (SEE NOTE 2)

- ASSUME $V_{BEND} = \frac{V_{CULVERT} + \left[\frac{V_{ENTRANCE} + V_{CULVERT}}{2}\right]}{2} = \frac{\frac{3}{8} V_{CULVERT} + \left[\frac{46.6}{14^2} \times \frac{1}{3} \times V_{CULVERT}\right]}{2}$

(AVG. VELOCITY IN TRANSITION)

NOTE 2: FROM ELEMENTARY FLUID MECHANICS, 4TH EDITION, J.K. VENNARD, JOHN WILEY AND SONS, INC., NEW YORK, 1961, pp 317-318.

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$$V_{BEND} \simeq 0.81\ V_{CULVERT} = 0.81\ V_{TW}$$

$$H_B = 0.5\ \frac{(0.81\ V_{TW})^2}{2g} \simeq \underline{0.33\ \frac{V_{TW}^2}{2g}}$$

c) Loss due to Friction:

$$H_F = \left[\frac{29 n^2 L_{INLET}}{R_{INLET}^{4/3}}\right]\frac{V_{INLET}^2}{2g} + \left[\frac{29 n^2 L_{CULVERT}}{R_{CULVERT}^{4/3}}\right]\frac{V_{CULVERT}^2}{2g}$$

(Ref 19, p. 5-6)

WHERE n = MANNING'S ROUGHNESS COEFFICIENT = 0.012, (Ref 19, p. 5-30)
L = LENGTH, IN FT,
R = HYDRAULIC RADIUS, IN FT.

$L_{INLET} \simeq \underline{27}$ FT (AVG.) (FIG. 4)

$R_{INLET} = \frac{1}{2}\left[R_{ENTRANCE} + R_{CULVERT}\right]$

$= \frac{1}{2}\left[\frac{(14)(64)}{(4)(14)} + \frac{(46.6)}{(25.4)}\right] = \underline{2.67}$ FT

$V_{INLET} \simeq \frac{V_{ENTRANCE} + V_{CULVERT}}{2} = 0.62\ V_{CULVERT}$ (SEE ABOVE)

$L_{CULVERT} \simeq \underline{55}$ FT (FIG. 4)

$R_{CULVERT} \simeq \frac{(46.6)}{(25.4)} = \underline{1.83}$ FT

$$\therefore H_F = \left[\frac{(29)(0.012)^2(27)}{(2.67)^{4/3}}\right]\frac{(0.62\ V_{TW})^2}{2g} + \left[\frac{(29)(0.012)^2(55)}{(1.83)^{4/3}}\right]\frac{V_{TW}^2}{2g}$$

$$= \underline{0.11\ \frac{V_{TW}^2}{2g}}$$

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$$H_L = H_E + H_B + H_F$$
$$= \left[0.03 + 0.33 + 0.11 \right] \frac{V_{TW}^2}{2g}$$
$$= \underline{0.47 \frac{V_{TW}^2}{2g}}$$

- RE-WRITE ENERGY BALANCE:

$$Z_R = Z_{TW} + \frac{V_{TW}^2}{2g} + H_L$$
$$Z_R = Z_{TW} + \frac{V_{TW}^2}{2g} + 0.47 \frac{V_{TW}^2}{2g}$$
$$\underline{Z_R = Z_{TW} + 1.47 \frac{V_{TW}^2}{2g}}$$

CALCULATE TAILWATER DEPTHS:

ESTIMATE TAILWATER DEPTH AT SECTION JUST DOWNSTREAM OF OUTLET (CONSERVATIVE). SECTION IS RECTANGULAR, WITH 11 FT HIGH CONCRETE SIDEWALLS AND 7' BOTTOM WIDTH. ASSUME THAT DISCHARGE WITHIN CHANNEL IS FROM CULVERT ONLY (OVERTOPPING DISCHARGE NOT INCLUDED), AND MAXIMUM DEPTH WITHIN CHANNEL IS TO TOP OF SIDEWALLS (SEE PHOTOS 4, 10). USE MANNING'S EQUATION:

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \qquad \text{(REF 19, p. 5-12)}$$

WHERE Q = DISCHARGE, IN CFS,
A = FLOW AREA, IN FT^2,
R = HYDRAULIC RADIUS, IN FT,
S = CHANNEL SLOPE, IN FT/FT,
n = ROUGHNESS COEFFICIENT = 0.020 (COMPOSITE ESTIMATE).

SUBJECT DAM SAFETY INSPECTION
WADHAM CREEK DAM

BY DJS DATE 7-21-80 PROJ. NO. 79-273-547

CHKD. BY WJV DATE 8-4-80 SHEET NO. 12 OF 70

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$A = 7 \times y$

$R = A/P = \frac{7y}{7+2y}$

$S = \frac{576-575}{39} = 0.026$ FT/FT (FIG. 4)

$Q = \left(\frac{1.49}{.020}\right)(7y)\left(\frac{7y}{7+2y}\right)^{2/3}\sqrt{.026}$

$Q = 12.0\,(7y)\left(\frac{7y}{7+2y}\right)^{2/3}$

OUTLET CONTROL RATING CURVE:

TAILWATER ELEVATION (FT)	y_{TW} ① (FT)	Q ② (CFS)	V_{TW} ③ (FPS)	$1.47\frac{V_{TW}^2}{2g}$ (FT)	Z_R ④ (FT)
584.0	8.1	1235	26.5	16.0	600.0
584.5	8.6	1326	28.5	18.5	603.0
585.0	9.1	1418	30.4	21.1	606.1
585.3	9.4	1474	31.6	22.8	608.1
585.5	9.6	1511	32.4	24.0	609.5
585.7	9.8	1548	33.2	25.2	610.9
585.9	10.0	1585	34.0	26.4	612.3

① y_{TW} = TW ELEV − 575.9
② $Q = 12.0\,(7y)\left(\frac{7y}{7+2y}\right)^{2/3}$
③ $V_{TW} = Q/46.6$
④ Z_R = TW ELEV + $1.47\frac{V_{TW}^2}{2g}$

IT CAN BE SEEN HERE THAT THE OUTLET CONTROL FLOWS ARE LOWER THAN THE INLET CONTROL FLOWS (ON SHEET 8) FOR CORRESPONDING ELEVATIONS. THUS THE OUTLET CONTROL FLOWS DICTATE.

JBJECT DAM SAFETY INSPECTION
WADHAM CREEK DAM

BY DTS DATE 7-21-80 PROJ. NO. 79-227-547

CHKD. BY WJV DATE 7-21-80 SHEET NO. 13 OF —

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PRINCIPAL SPILLWAY RATING CURVE :

RESERVOIR ELEVATION (FT)	Q* (CFS)		RESERVOIR ELEVATION (FT)	Q* (CFS)
599.0	0	(LOW TOP OF DAM)	605.5	1400
600.0	130		606.0	1420
601.0	360		606.5	1430
602.0	620		607.0	1440
602.5	760		607.5	1460
603.0	910		608.0	1470
603.5	1050		608.5	1480
604.0	1190		609.0	1500
604.5	1330		609.5	1510
605.0	1390		610.0	1520

* FROM TABLE - SHEET 6, UP TO EL. 604.5,
FROM TABLE - SHEET 12, EL. 605.0 AND ABOVE (INTERPOLATED).

SUBJECT DAM SAFETY INSPECTION
WADHAM CREEK DAM

BY DTS DATE 7-21-82 PROJ. NO. 79-203-547

CHKD. BY WJV DATE 7-21-82 SHEET NO. 14 OF 21

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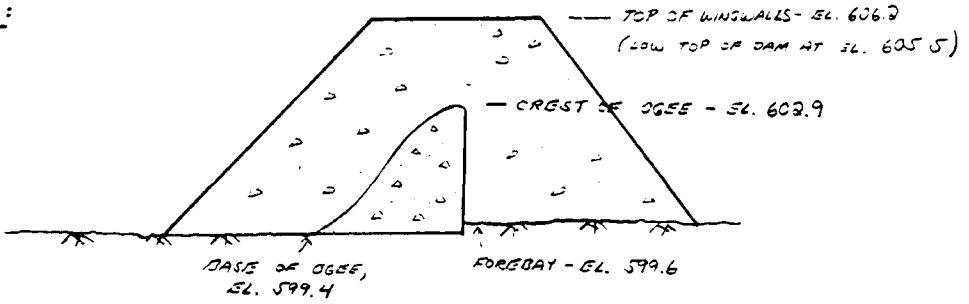
CAPACITY OF EMERGENCY SPILLWAY

PROFILE:



THE EMERGENCY SPILLWAY CONSISTS OF A CONCRETE OGEE WEIR STRUCTURE WHICH DISCHARGES INTO A RECREATIONAL AREA ADJACENT TO THE EMBANKMENT. DISCHARGE OVER THE WEIR CAN BE ESTIMATED BY THE RELATIONSHIP

$$Q = CLH^{3/2} \qquad \text{(REF 4, p. 373)}$$

WHERE Q = WEIR DISCHARGE, IN CFS,
C = COEFFICIENT OF DISCHARGE,
L = LENGTH OF WEIR CREST, IN FT,
H = WEIR HEAD, IN FT.

THE DESIGN HEAD IS ASSUMED TO BE 3.3 FEET, OR AT THE TOP OF THE SPILLWAY WINGWALLS. THE RELATIONSHIPS FOR OGEE-TYPE WEIRS, GIVEN IN REF 4, pp. 372-382, WILL BE USED HERE. FOR A FOREBAY DEPTH = 3.3 FT,

$$\frac{P}{H_o} = \frac{3.3}{3.3} = 1.0$$

$$\therefore C_o = 3.88 \qquad \text{(REF 4, FIG. 249)}$$

AS THE HEAD ON THE WEIR BECOMES SMALL, DISCHARGE IS REDUCED DISPROPORTIONATELY, DUE TO THE ROUGHNESS AND THE CONTACT PRESSURE BETWEEN

SUBJECT DAM SAFETY INSPECTION
WADHAM CREEK DAM

BY DJS DATE 7-30-80 PROJ. NO. 79-202-547

CHKD. BY WJV DATE 8-4-80 SHEET NO. 15 OF 20

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THE WATER AND THE WEIR SURFACE. THUS, THE DISCHARGE COEFFICIENT (C) TAKES ON A LOWER VALUE THAN THAT OF DESIGN HEAD. THE OPPOSITE TREND OCCURS FOR HEADS GREATER THAN THAT OF DESIGN. THEREFORE, THE DISCHARGE COEFFICIENT WILL BE MODIFIED APPROPRIATELY, ACCORDING TO REF 4, FG. 253.

THE SPILLWAY DISCHARGES INTO A RELATIVELY WIDE AND FLAT AREA ADJACENT THE DAM. IMMEDIATELY DOWNSTREAM OF THE WEIR (≈ 50 FT) IS A FENCE (SEE PHOTOGRAPH 7), WHICH COULD CONCEIVABLY ACCUMULATE DEBRIS SHOULD THE EMERGENCY SPILLWAY DISCHARGE UNDER HIGH INTENSITY FLOWS. THIS ACCUMULATION OF DEBRIS COULD RESULT IN TAILWATER LEVELS WHICH COULD RADICALLY ALTER THE DISCHARGE CAPABILITIES OF THE SPILLWAY. FOR THIS ANALYSIS, THEN, IT WILL BE ASSUMED THAT THE FENCE IS REMOVED, OR DOES NOT INTERFERE WITH EMERGENCY SPILLWAY DISCHARGES. SHOULD THE FENCE BE REMOVED, THE TAILWATER LEVELS WITHIN THE FLAT OPEN AREA (AVG. SLOPE ≈ 1.1%) SHOULD NOT INTERFERE WITH WEIR DISCHARGES.

IT WILL ALSO BE ASSUMED THAT APPROACH CHANNEL LOSSES ARE INSIGNIFICANT, AND MAY BE IGNORED.

THE EMERGENCY SPILLWAY RATING CURVE IS GIVEN ON SHEET 16.

SUBJECT DAM SAFETY INSPECTION
WADHAM CREEK DAM

BY DCS DATE 7-30-80 PROJ. NO. 79-203-547

CHKD. BY WJV DATE 8-4-80 SHEET NO. 16 OF 78

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EMERGENCY SPILLWAY RATING TABLE

	RESERVOIR ELEVATION (FT)	H (FT)	H/H_o	C/C_o ①	C ②	Q ③ (CFS)
	602.9	—	—	—	—	0
	603.0	0.1	0.03	0.80	3.10	10
	603.5	0.6	0.18	0.85	3.30	160
	604.0	1.1	0.33	0.89	3.45	410
	604.5	1.6	0.48	0.92	3.57	740
	605.0	2.1	0.64	0.95	3.69	1150
(LOW TOP OF DAM)	605.5	2.6	0.79	0.97	3.76	1610
	606.0	3.1	0.94	0.99	3.84	2140
(TOP OF SPWY WINGWALLS)	606.2	3.3	1.00	1.00	3.88	2370
	606.5	3.6	1.09	1.01	3.92	2730
	607.0	4.1	1.24	1.03	4.00	3390
	607.5	4.6	1.39	1.05	4.07	4100
	608.0	5.1	1.55	1.06	4.11	4830
	608.5	5.6	1.70	1.07	4.15	5610
	609.0	6.1	1.85	1.07	4.15	6380
	609.5	6.6	2.00	1.07	4.15	7180
	610.0	7.1	2.15	1.07	4.15	8010

① FROM REF 4, FIG. 250, p. 378.

② $C = C/C_o \times C_o = C/C_o \times 3.88$

③ $Q = CLH^{3/2}$, $L = 100$ FT

SUBJECT DAM SAFETY INSPECTION
WADHAM CREEK DAM

BY DJS DATE 2-30-80 PROJ. NO. 79-203-547

CHKD. BY WJI DATE 2-4-80 SHEET NO. 17 OF 20

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EMBANKMENT RATING CURVE

ASSUME THAT THE EMBANKMENT BEHAVES ESSENTIALLY AS A BROAD-CRESTED WEIR WHEN OVERTOPPING OCCURS. THUS, THE DISCHARGE CAN BE ESTIMATED BY THE RELATIONSHIP

$$Q = CLH^{3/2} \qquad \text{(REF 5, p. 5-23)}$$

WHERE Q = DISCHARGE OVER EMBANKMENT, IN CFS,
L = LENGTH OF EMBANKMENT OVERTOPPED, IN FT,
H = HEAD, IN FT; IN THIS CASE IT IS THE AVERAGE "FLOW AREA WEIGHTED" HEAD ABOVE THE LOW TOP OF DAM;
C = COEFFICIENT OF DISCHARGE, DEPENDENT UPON THE HEAD AND THE WEIR BREADTH.

LENGTH OF EMBANKMENT INUNDATED VS. RESERVOIR ELEVATION.

RESERVOIR ELEVATION (FT)	EMBANKMENT LENGTH (FT)	
605.5	0	
605.7	40	
605.8	160	
605.9	255	
606.0	305	
606.2	360	
606.5	365	
607.0	370	
607.5	375	
608.0	380	(FROM FIELD SURVEY AND FIG. 2)
609.0	395	RT. ABUTMENT SS = 9H:1V
610.0	405	LT. ABUTMENT SS = 3H:1V

SUBJECT DAM SAFETY INSPECTION

WADHAM CREEK DAM

BY DJS DATE 2-23-80 PROJ. NO. 23-395-547

CHKD. BY WJV DATE 2-4-80 SHEET NO. 18 OF 22

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ASSUME THAT INCREMENTAL DISCHARGES OVER THE EMBANKMENT FOR SUCCESSIVE RESERVOIR ELEVATIONS ARE APPROXIMATELY TRAPEZOIDAL IN CROSS-SECTIONAL FLOW AREA. THEN ANY INCREMENTAL AREA OF FLOW CAN BE ESTIMATED AS $H_i [(L_1 + L_2)/2]$, WHERE L_1 = LENGTH OF EMBANKMENT OVERTOPPED AT HIGHER ELEVATION, L_2 = LENGTH AT LOWER ELEVATION, H_i = DIFFERENCE IN ELEVATIONS. THUS, THE TOTAL AVERAGE "FLOW AREA WEIGHTED" HEAD CAN BE ESTIMATED AS H_w = (TOTAL FLOW AREA / L_1).

EMBANKMENT RATING TABLE:

RESERVOIR ELEVATION (FT)	L_1 (FT)	L_2 (FT)	INCREMENTAL HEAD, H_i (FT)	INCREMENTAL FLOW AREA, A_i (FT²) ①	TOTAL FLOW AREA, A_T (FT²)	WEIGHTED HEAD, H_w (FT) ②	H_w/L ③	C ④	Q (CFS) ⑤
605.5	0	—	—	—	—	—	—	—	0
605.7	40	0	0.2	4	4	0.1	0.01	2.93	0
605.8	160	40	0.1	10	14	0.1	0.01	2.93	10
605.9	255	160	0.1	21	35	0.1	0.01	2.93	30
606.3	305	255	0.1	28	63	0.2	0.03	2.97	80
606.2	360	305	0.2	67	130	0.4	0.05	3.01	270
606.5	365	360	0.3	109	239	0.7	0.09	3.03	650
607.0	370	365	0.5	184	423	1.1	0.14	3.04	1330
607.5	375	370	0.5	186	609	1.6	0.20	3.08	2340
608.0	380	375	0.5	189	798	2.1	0.26	3.09	3570
609.0	395	380	1.0	388	1186	3.0	0.38	3.09	6340
610.0	405	395	1.0	400	1586	3.9	0.49	3.09	9640

① $A_i = H_i \left[\frac{(L_1 + L_2)}{2} \right]$

② $H_w = A_T / L_1$

③ l = BREADTH OF CREST = 8 FT (FIELD MEASURED)

④ $C = A(H, l)$, FROM REF 12, FIG. 24.

⑤ $Q = C L_1 H_w^{3/2}$

SUBJECT DAM SAFETY INSPECTION
WADHAM CREEK DAM

BY JJC DATE 7-30-80 PROJ. NO. 79-203-547

CHKD. BY WJV DATE 8-4-80 SHEET NO. 19 OF 22

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TOTAL FACILITY RATING CURVE

$$Q_{TOTAL} = Q_{PRINCIPAL\ SPILLWAY} + Q_{EMERGENCY\ SPILLWAY} + Q_{EMBANKMENT}$$

	RESERVOIR ELEVATION (FT)	$Q_{PRINCIPAL\ SPILLWAY}$ ① (CFS)	$Q_{EMERGENCY\ SPILLWAY}$ ② (CFS)	$Q_{EMBANKMENT}$ ③ (CFS)	Q_{TOTAL} (CFS)
(TOP OF SERVICE SPILLWAY RISER)	599.0	0	—	—	0
	600.0	130	—	—	130
	601.0	360	—	—	360
	602.0	620	—	—	620
(CREST OF EMERGENCY SPILLWAY)	602.9	880 *	0	—	880
	603.0	910	10	—	920
	603.5	1050	160	—	1210
	604.0	1190	410	—	1600
	604.5	1330	740	—	2070
	605.0	1390	1150	—	2540
(LOW TOP OF DAM)	605.5	1400	1610	0	3010
	605.9	1420	2030 *	20	3470
	606.0	1420	2140	80	3640
	606.2	1420	2370	270	4060
	606.5	1430	2730	650	4810
	607.0	1440	3390	1300	6130
	607.5	1460	4100	2340	7900
	608.0	1470	4830	3570	9870
	609.0	1500	6380	6340	14,220
	610.0	1520	8010	9640	19,170

* BY LINEAR INTERPOLATION.

① FROM SHEET 13.

② FROM SHEET 16.

③ FROM SHEET 18.

SUBJECT DAM SAFETY INSPECTION
WADHAM CREEK DAM

BY DJS DATE 7-31-80 PROJ. NO. 79-202-547

CHKD. BY WJ/ DATE 8-4-80 SHEET NO. 30 OF 36

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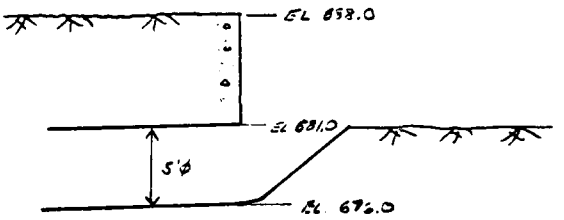
DUFFEY RUN DIVERSION

DUFFEY RUN DEBRIS DAM IS LOCATED IN THE WATERSHED ADJACENT TO THE WADHAM CREEK BASIN. APPROXIMATELY 300 FEET DOWNSTREAM FROM THIS DEBRIS DAM IS A DIVERSION STRUCTURE, CONSISTING OF A 5-FOOT DIAMETER CONCRETE CULVERT AND A DOWNSTREAM RECTANGULAR CONCRETE CHANNEL, WHICH DRAINS INTO THE WADHAM CREEK BASIN JUST UPSTREAM OF THE DAM. DISCHARGE FROM THIS STRUCTURE MUST BE INCLUDED IN THE WADHAM CREEK RESERVOIR INFLOW COMPUTATION.

PROFILE OF INLET:



(FROM FIELD OBSERVATION AND DESIGN DRAWINGS NOT INCLUDED IN APPENDIX E.)

THE DRAINAGE AREA OF THE DUFFEY RUN DEBRIS DAM IS APPROXIMATELY 0.3 SQUARE MILES (SEE NOTE 3). PMF OUTFLOWS FOR THIS BASIN ARE EXPECTED TO BE ON THE ORDER OF 800-1100 CFS. THEREFORE, HEADWATER DEPTHS AT THE CULVERT OF 12-15 FT DO NOT SEEM UNREASONABLE FOR EVENTS OF THIS MAGNITUDE. ASSUMING INLET CONTROL AND A HEADWATER DEPTH OF 15 FT, THE CAPACITY OF THE CULVERT IS APPROXIMATELY 340 CFS (REF 19, CHART 2, HEADWALL AND SQUARE EDGED INLET). THEREFORE, TO BE CONSERVATIVE, A VALUE OF 340 CFS WILL BE ADDED TO THE WADHAM CREEK BASIN PEAK INFLOW (FOR EACH EVENT).

NOTE 3: FROM "DAMS, RESERVOIRS, AND NATURAL LAKES," WATER RESOURCES BULLETIN No. 5, COMMONWEALTH OF PENNSYLVANIA, DEPT. OF FORESTS AND WATER, HARRISBURG, PA, 1970.

SUBJECT DAM SAFETY INSPECTION
WADHAM CREEK DAM
BY DJS DATE 8-2-80 PROJ. NO. 79-203-547
CHKD. BY WJV DATE 8-4-80 SHEET NO. A OF 5

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OVERTOPPING ANALYSIS

SUMMARY INPUT/OUTPUT SHEETS

```
DAM SAFETY INSPECTION
WADHAM CREEK DAM **** OVERTOPPING ANALYSIS ****
15-MINUTE TIME STEP AND 72-HOUR STORM DURATION

                    JOB SPECIFICATION
   NO     NHR    NMIN    IDAY     IHR    IMIN   METRC    IPLT    IPRT   NSTAN
  280       0      15       0       0       0       0       0       0       0
                       JOPER     NWT   LROPT   TRACE
                           5       0       0       0

          MULTI-PLAN ANALYSES TO BE PERFORMED
              NPLAN= 1 NRTIO= 3 LRTIO= 1
     RTIOS=     .80     .90    1.00

**********      **********      **********      **********      **********

                SUB-AREA RUNOFF COMPUTATION

     INFLOW INTO RESERVOIR

              ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                  1       0       0       0       0       0       1       0       0

                    HYDROGRAPH DATA
   IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO  ISNOW  ISAME  LOCAL
       1       1    1.00    0.00    1.00    0.00   0.000      0      1      0

                    PRECIP DATA
            SPFE     PMS      R6     R12     R24     R48     R72     R96
            0.00   22.20  117.50  127.00  136.00  142.50  145.00    0.00
TRSPC COMPUTED BY THE PROGRAM IS  .800

                    LOSS DATA
  LROPT   STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
      0    0.00    0.00    1.00    0.00    0.00    1.00    1.00     .05    0.00    0.00

                UNIT HYDROGRAPH DATA
          TP=   .96    CP= .30    NTA=  0

                RECESSION DATA
          STRTQ=   -1.50    QRCSN=    -.05    RTIOR= 2.00
APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC= 3.71 AND R=10.56 INTERVALS

     UNIT HYDROGRAPH 58 END-OF-PERIOD ORDINATES, LAG=    .97 HOURS, CP=  .30  VOL= 1.00
  23.    86.   158.   195.   191.   174.   158.   144.   131.   119.
 108.    99.    90.    82.    74.    68.    61.    56.    51.    46.
  42.    38.    35.    32.    29.    26.    24.    22.    20.    18.
  16.    15.    13.    12.    11.    10.     9.     8.     8.     7.
   6.     6.     5.     5.     4.     4.     4.     3.     3.     3.
   3.     3.     2.     2.     2.     2.     1.     1.

   0                    END-OF-PERIOD FLOW
 MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS   COMP Q      MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS   COMP Q

                                                SUM  25.75  23.12   2.63   60085.
                                                     654.1( 587.1(  67.1(  1701.42)
```

INITIAL + CONSTANT RAINFALL LOSSES AS PER C.O.E.

BASE FLOW PARAMETERS AS PER C.O.E.

RESERVOIR INFLOW HYDROGRAPH (PMF)

		PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
	CFS	2197.	1691.	610.	209.	60088.
	CMS	68.	48.	17.	6.	1701.
	INCHES		15.73	22.69	23.29	23.29
	MM		399.47	576.28	591.56	591.56
	AC-FT		838.	1209.	1241.	1241.
	THOUS CU M		1034.	1492.	1531.	1531.

********** ********** ********** ********** **********

HYDROGRAPH ROUTING

ROUTE THROUGH RESERVOIR

ISTAO	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
101	1	0	0	0	0	1	0	0

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME	IOPT	IPMP	LSTR
0.0	0.000	0.00	1	1	0	0	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	0	0	0.000	0.000	0.000	-585.	-1

STAGE	599.00	600.00	601.00	602.00	602.90	603.00	603.50	604.00	604.50
	605.50	605.90	606.00	606.20	606.50	607.00	607.50	608.00	609.00
FLOW	0.00	130.00	360.00	620.00	880.00	920.00	1210.00	1600.00	2070.00
	3010.00	3470.00	3640.00	4060.00	4810.00	6130.00	7900.00	9870.00	14220.00
CAPACITY=	0.	0.	0.	1.	1.	2.	4.	4.	5.	7.
	R.	10.	12.	13.	16.	19.	22.			
ELEVATION=	585.	588.	590.	592.	594.	596.	598.	599.	600.	602.
	603.	604.	606.	606.	608.	610.	612.			

CREL	SPWID	COQW	EXPW	FLEVL	COOL	CAREA	EXPL
602.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0

DAM DATA

TOPEL	COQD	EXPD	DAMWID
605.5	0.0	0.0	0.

PEAK OUTFLOW IS 2400. AT TIME 40.75 HOURS

RESERVOIR OUTFLOW HYDROGRAPH. (PMF)

		PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
	CFS	2400.	1691.	610.	208.	59880.
	CMS	68.	48.	17.	6.	1696.
	INCHES		15.73	22.69	23.21	23.21
	MM		399.53	576.28	589.51	589.51
	AC-FT		838.	1209.	1237.	1237.
	THOUS CU M		1034.	1492.	1526.	1526.

SUBJECT DAM SAFETY INSPECTION

WADHAM CREEK DAM

BY DSS DATE 8-9-80 PROJ. NO. 79-203-547

CHKD. BY WJV DATE 8-4-80 SHEET NO. 8 OF 5

SUBJECT DAM SAFETY INSPECTION

BY DJS DATE 8-2-80 PROJ. NO. 12-283-547

CHKD. BY WJV DATE 8-4-80 SHEET NO. 5 OF 5

WADHAM CREEK DAM

SUMMARY OF DAM SAFETY ANALYSIS

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	585.00	602.90	605.50
STORAGE	0.	8.	12.
OUTFLOW	0.	880.	3010.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.80	604.34	0.00	10.	1919.	0.00	40.75	0.00
.90	604.59	0.00	11.	2159.	0.00	40.75	0.00
1.00	604.85	0.00	11.	2400.	0.00	40.75	0.00

PMF PEAK DISCHARGE = 2400 + 340 = 2740 CFS
(DUFFEY RUN DIVERSION)

MAX. W.S.ELEV. @PMF = 605.2 (BY LINEAR INTERPOLATION)

CONSULTANTS
Engineers • Geologists • Plan
Environmental Specialists

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APPENDIX E

FIGURES

LIST OF FIGURES

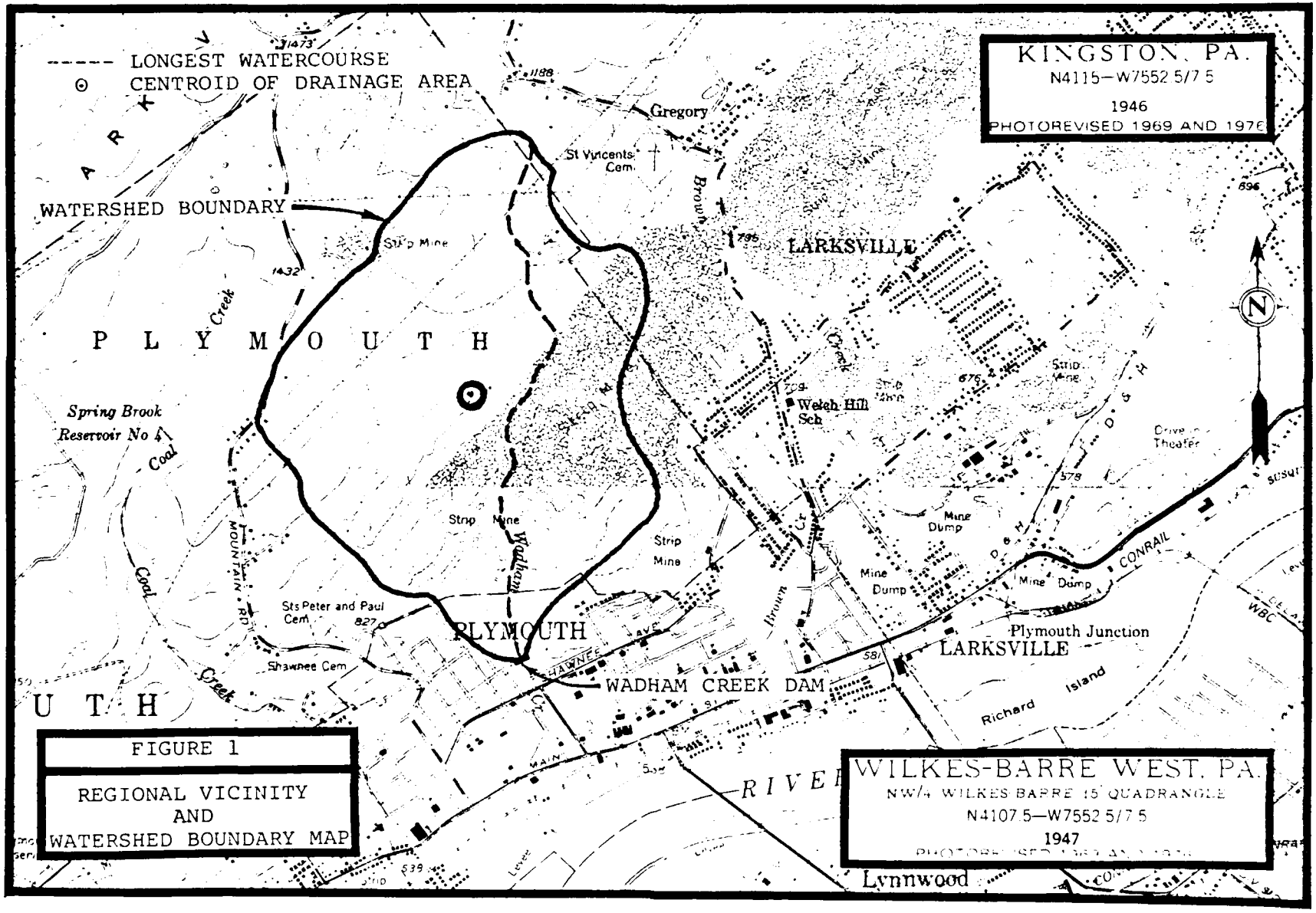
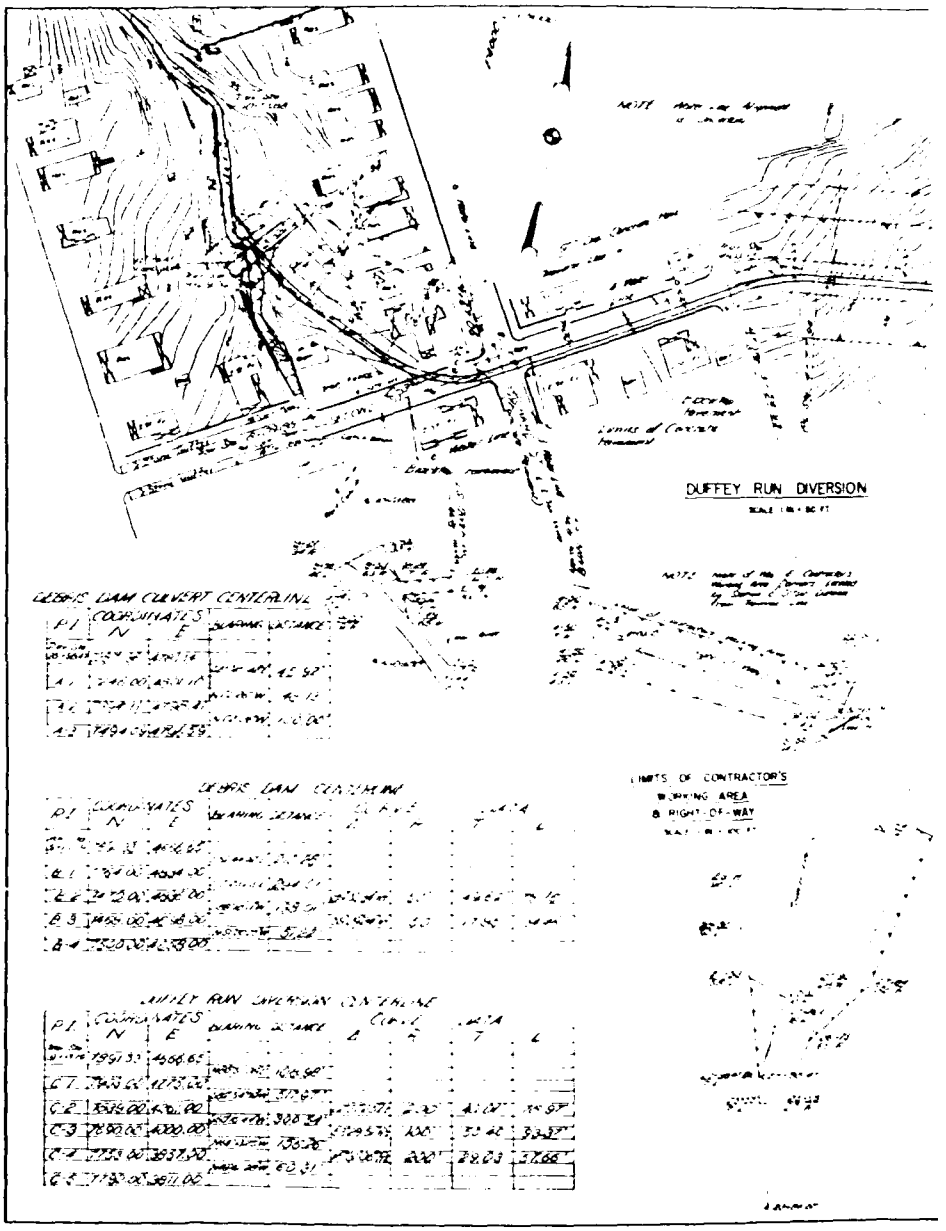


FIGURE 1

REGIONAL VICINITY
AND
WATERSHED BOUNDARY MAP


DUFFEY RUN DIVERSION
LIMITS OF CONTRACTOR'S
WORKING AREA
& RIGHT-OF-WAY

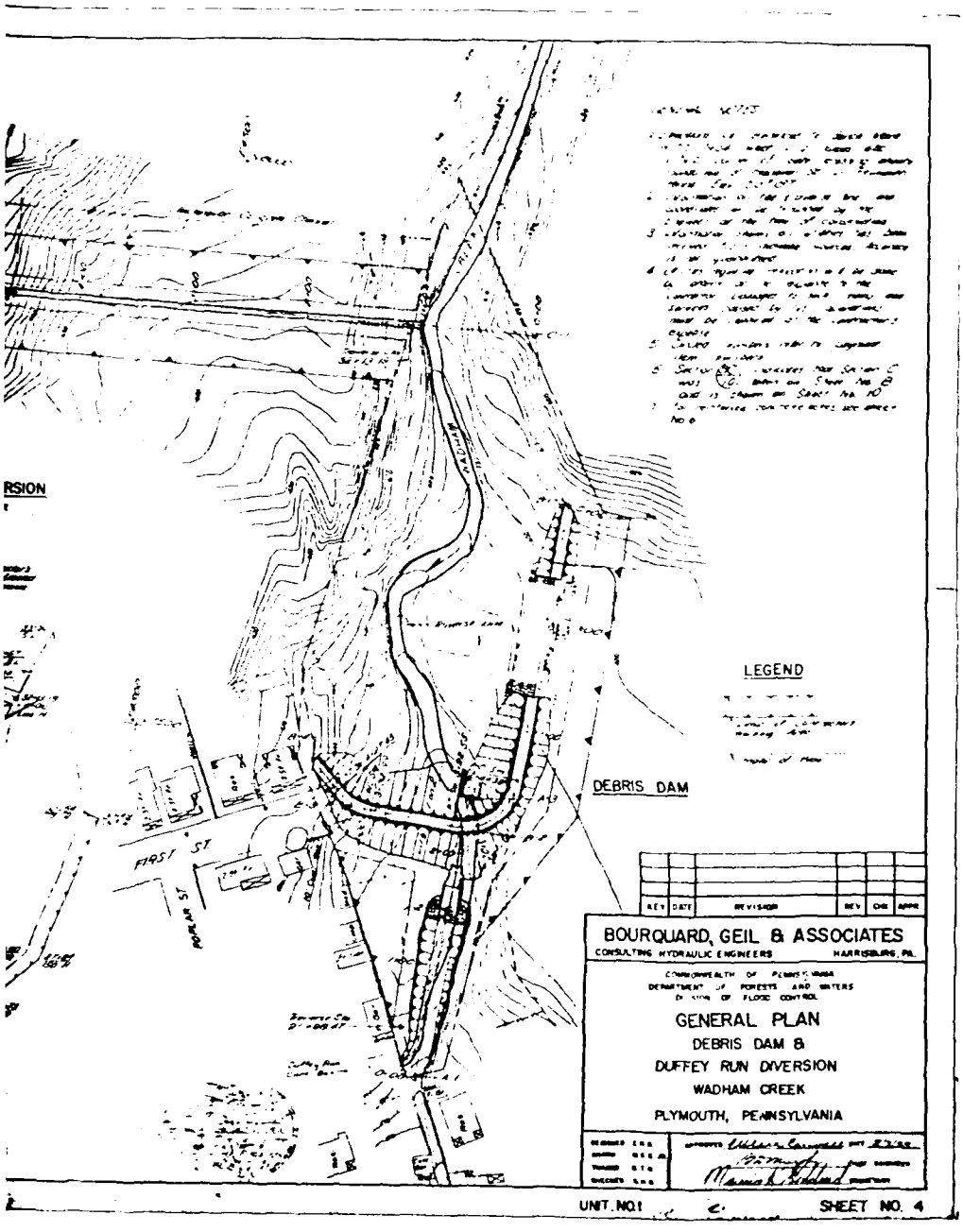


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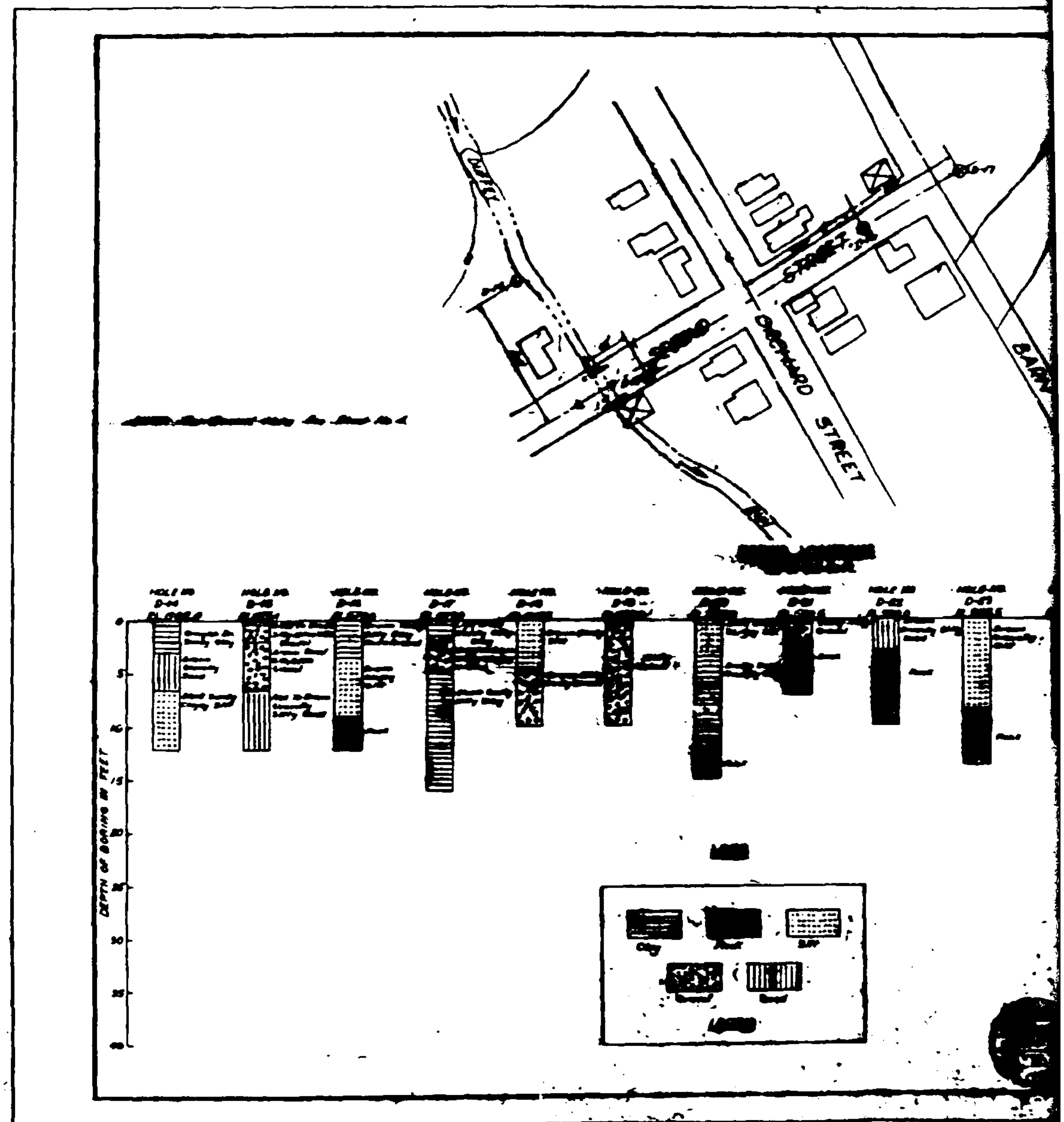
FIGURE 2


ORCHARD STREET
STREET
DEPTH OF BORING IN FEET

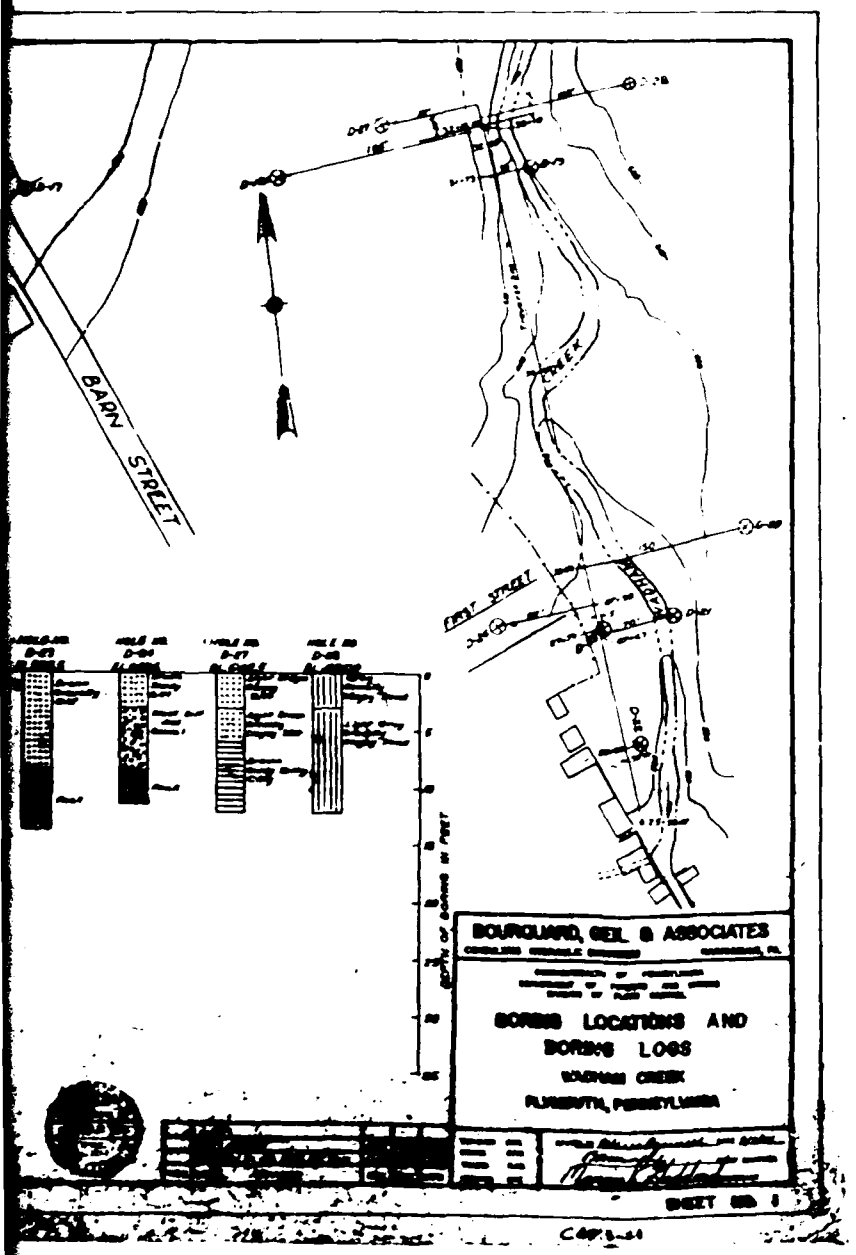


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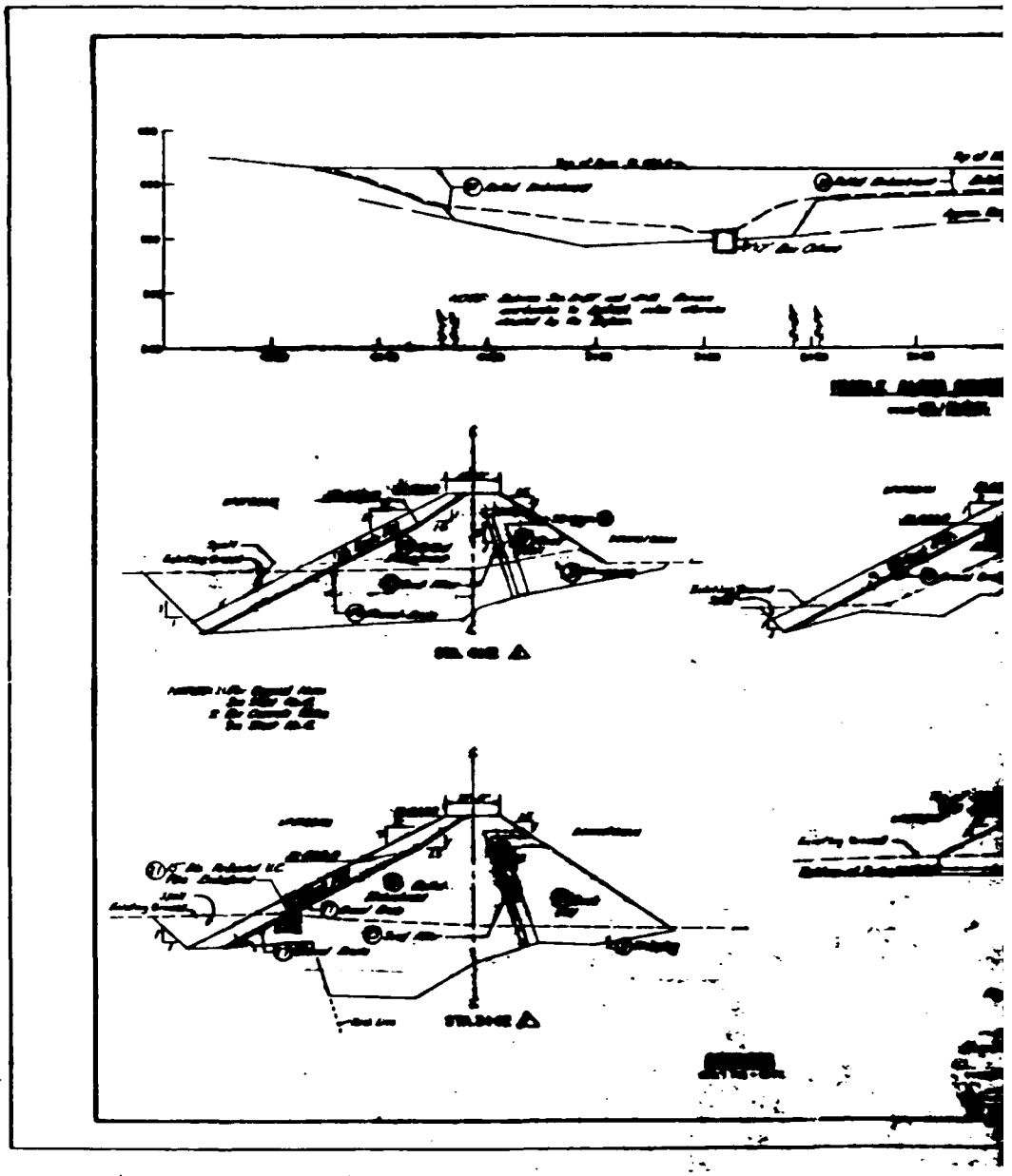
FIGURE 3



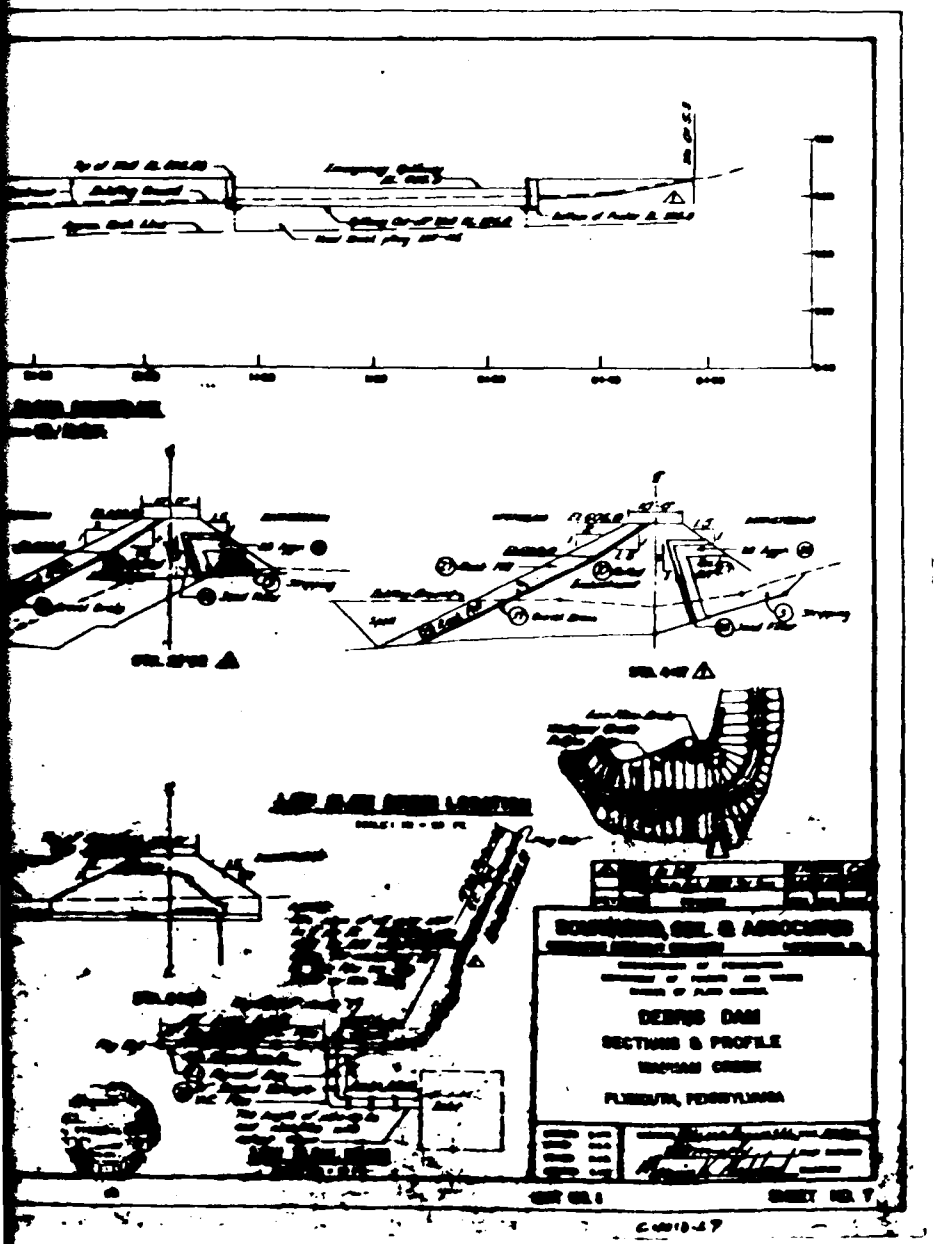


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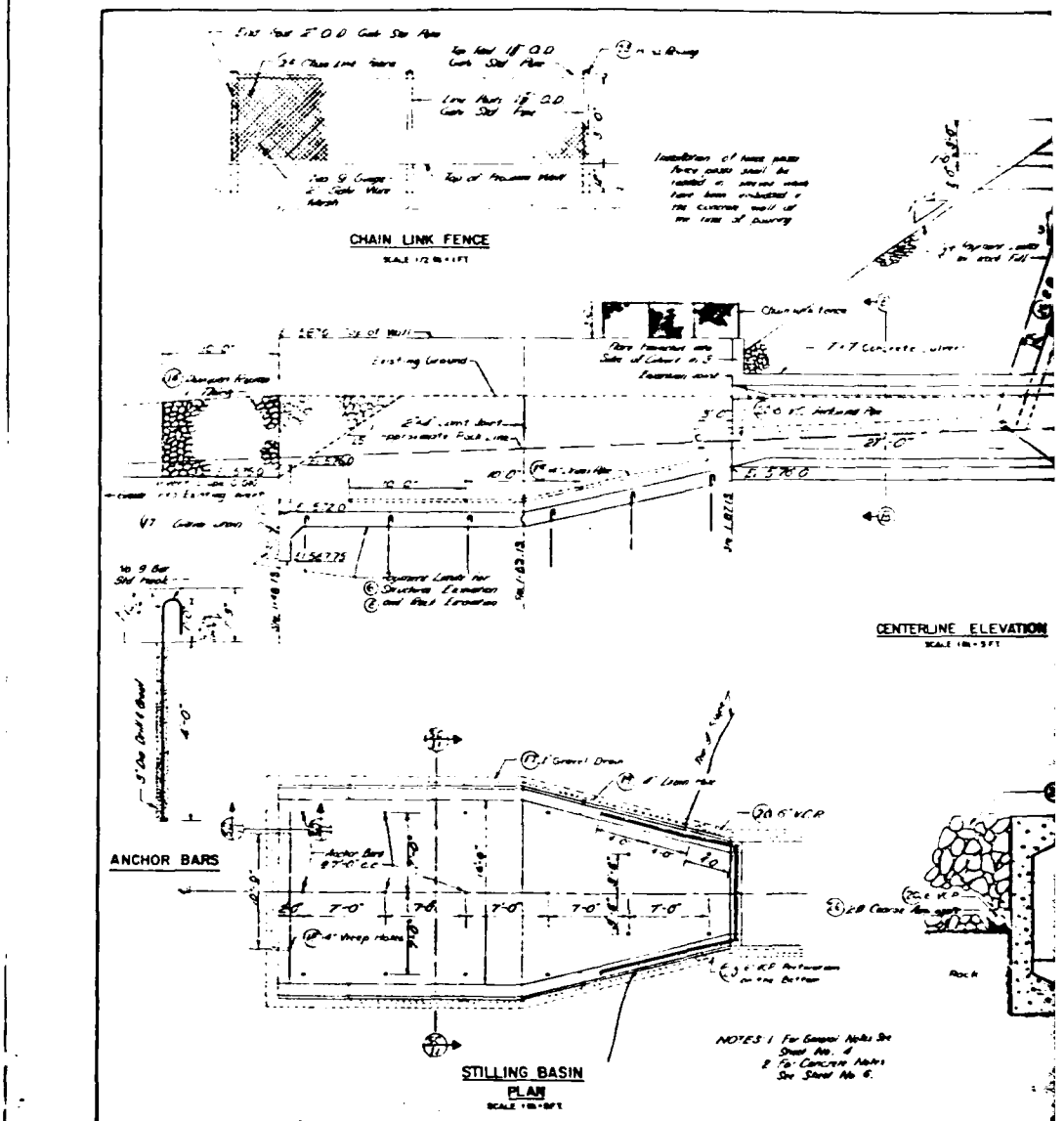
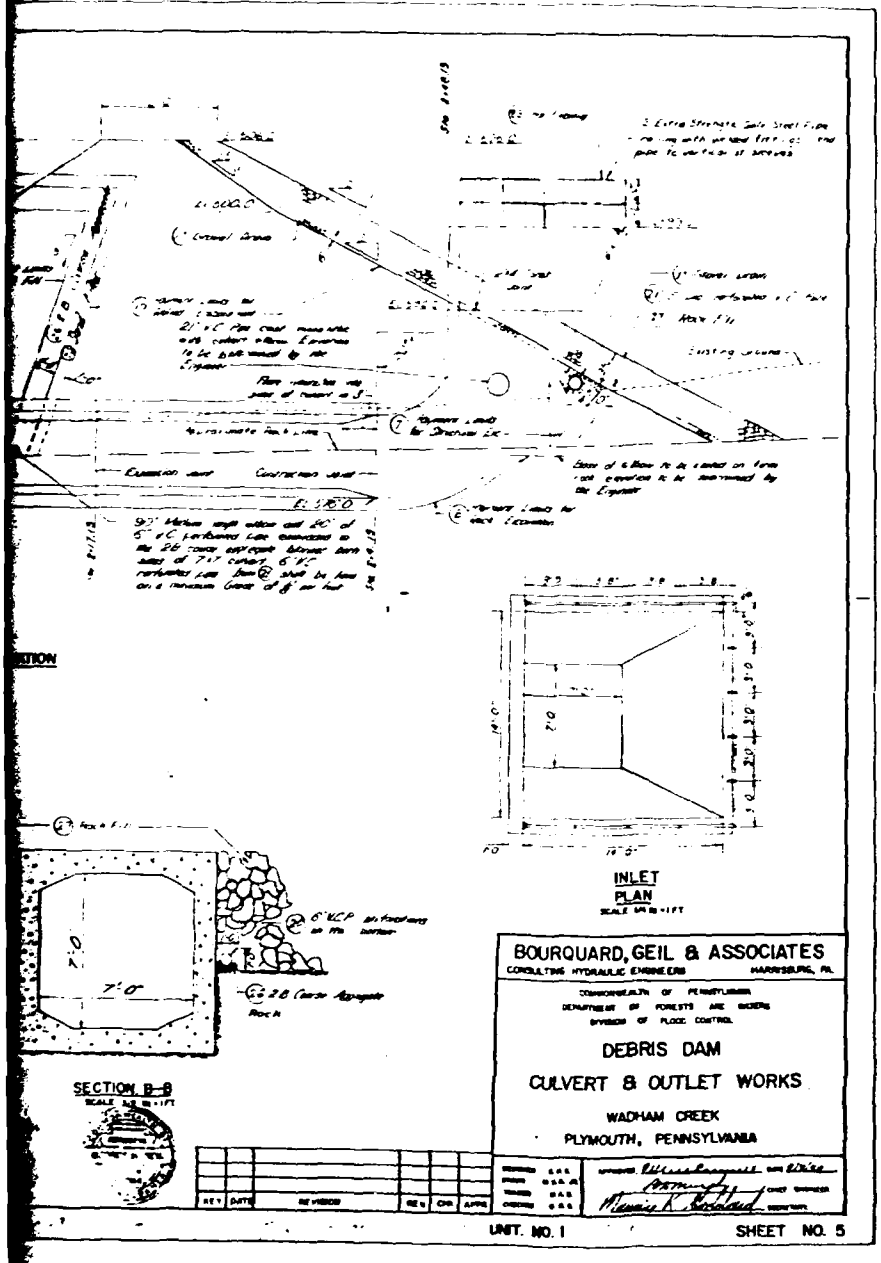
FIGURE 4


CHAIN LINK FENCE
CENTERLINE ELEVATION
ANCHOR BARS
STILLING BASIN
PLAN



2

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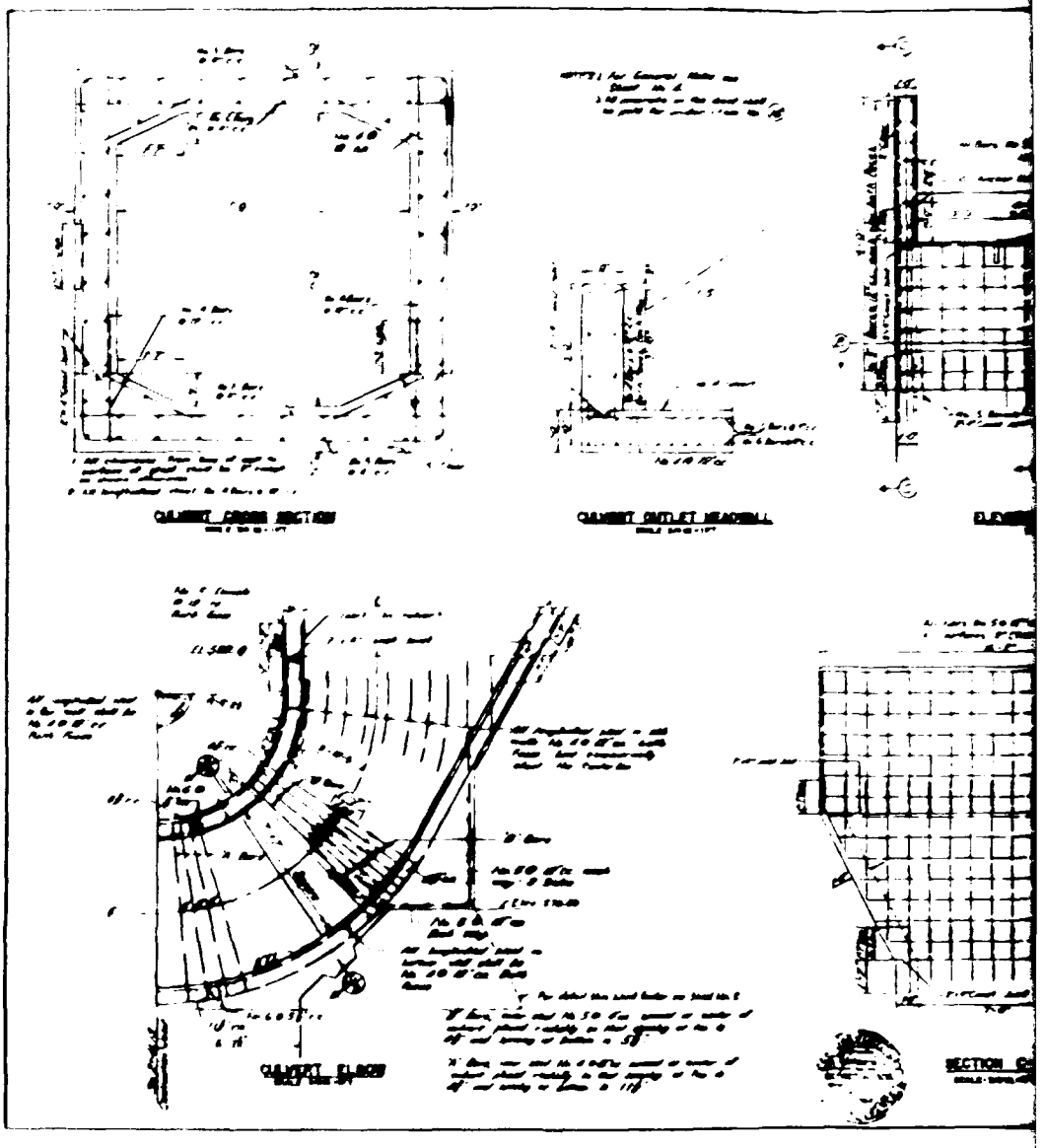
FIGURE 5



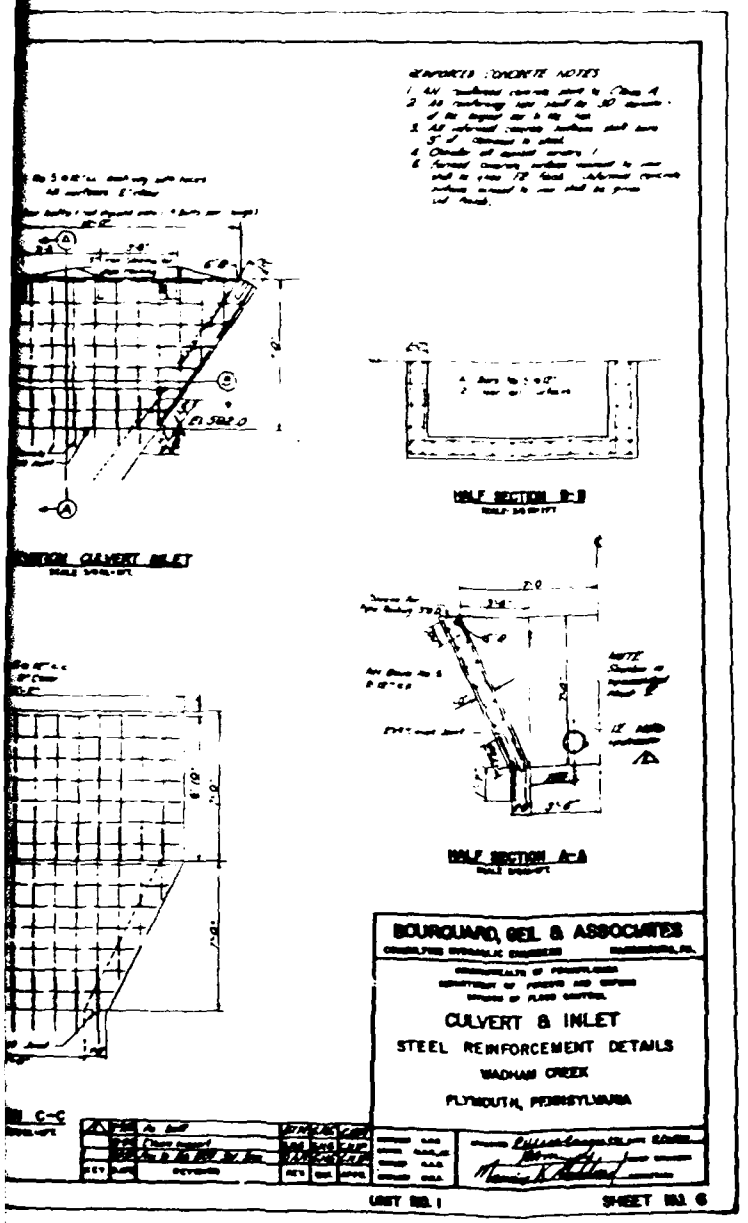


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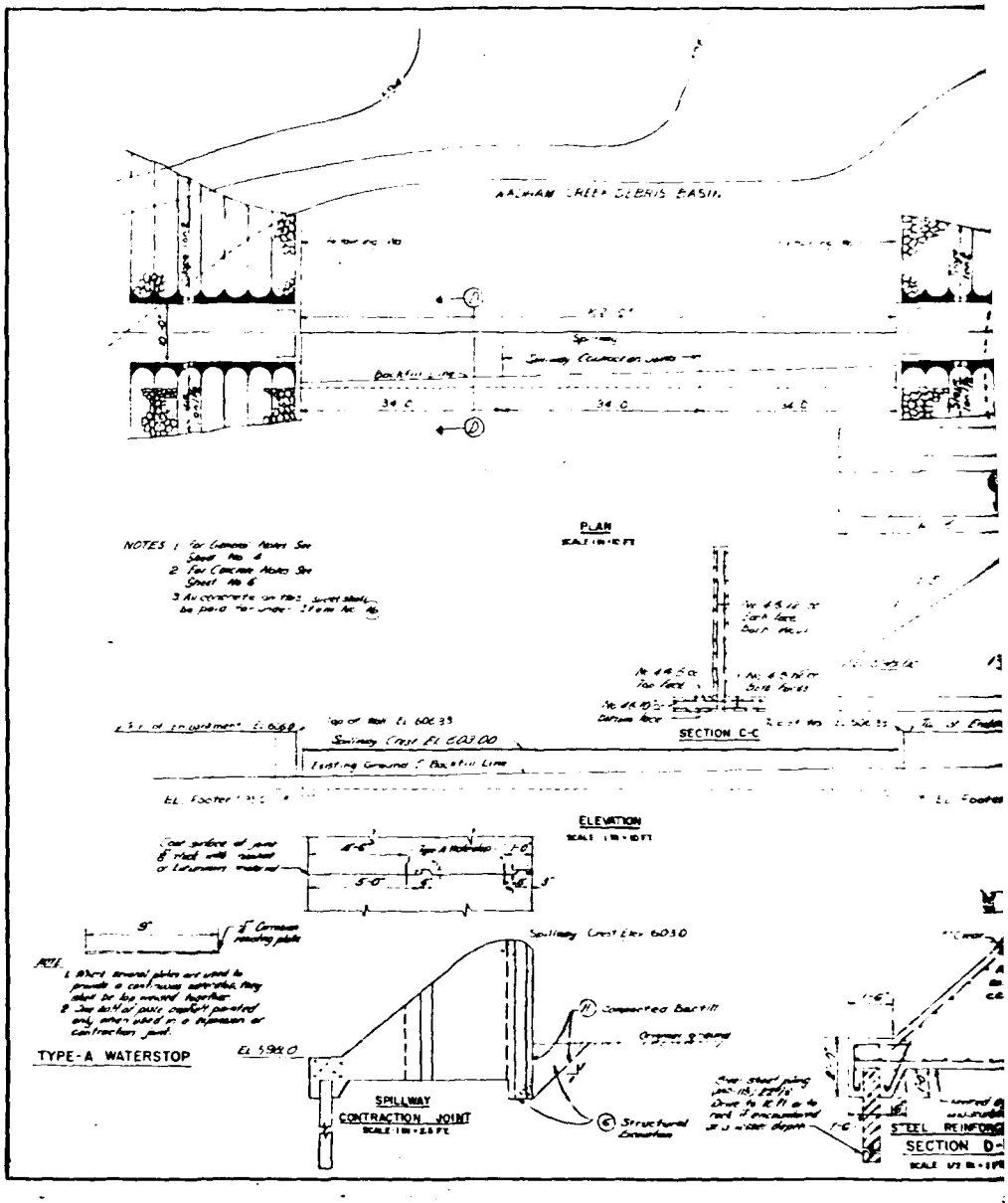
FIGURE 6


ALHAM CREEK DEBRIS BASIN
PLAN
SCALE IN = 10 FT
NOTES : 1 For Common Notes See Sheet No 4
2 For Concrete Notes See Sheet No 6
SECTION C-C
Spillway Crest El 6030.00
Existing Ground & Backfill Line
ELEVATION
SCALE 1 IN = 10 FT
TYPE-A WATERSTOP
NOTE
SPILLWAY CONTRACTION JOINT
SCALE 1 IN = 2.0 FT
Spillway Crest Elev 6030
Compacted Backfill
Original Ground
Structural Connection
STEEL REINFORC
SECTION D-

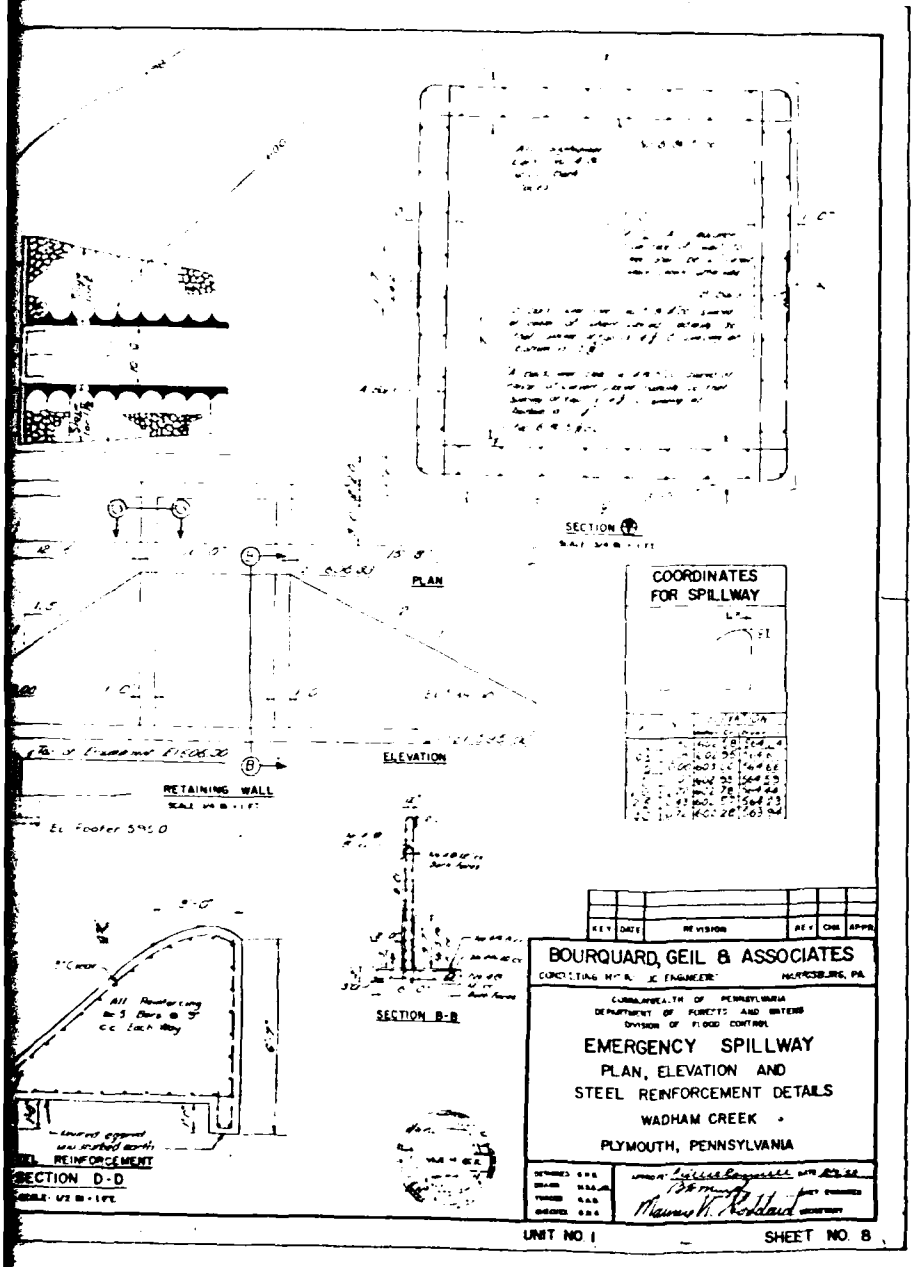


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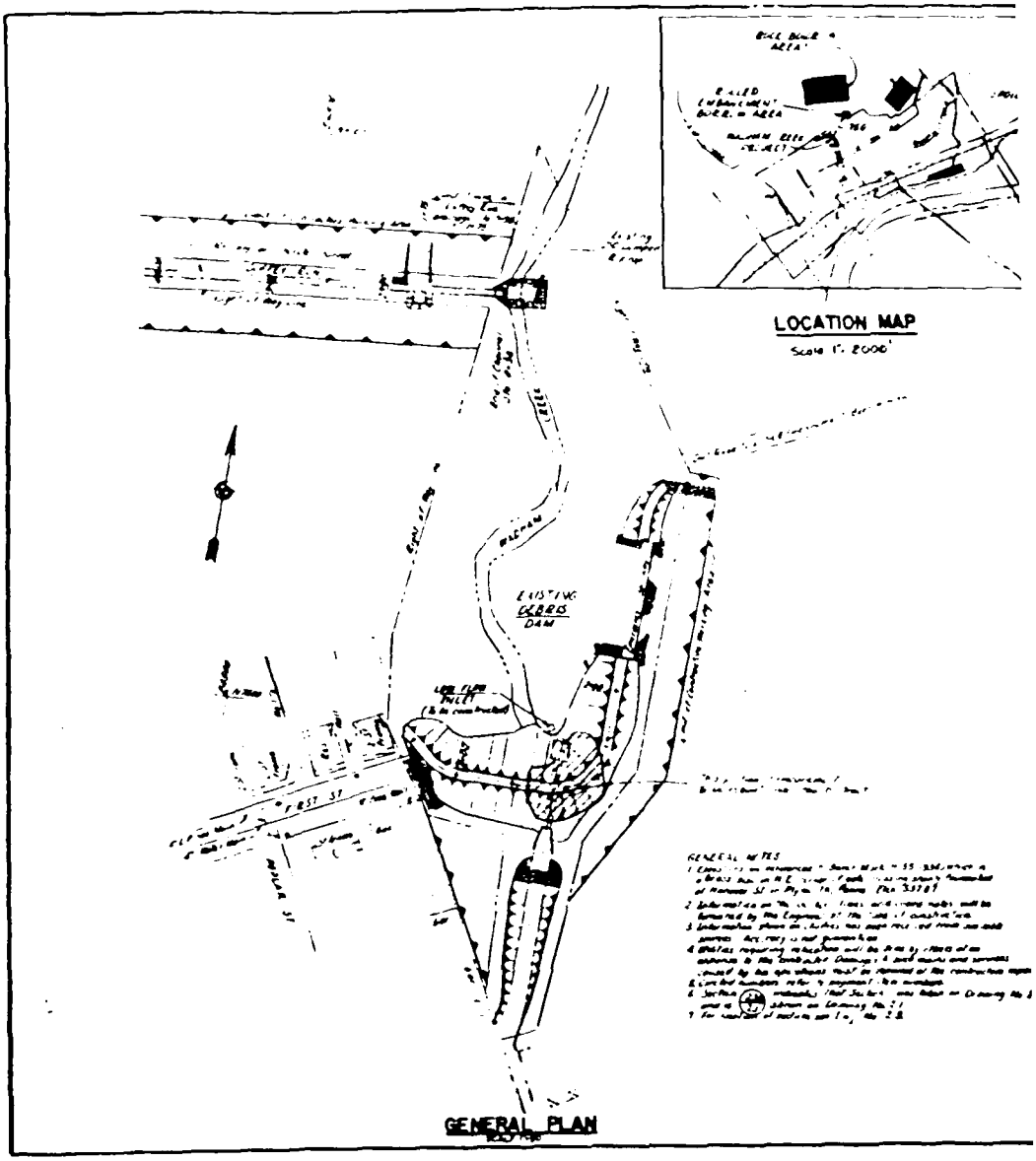
FIGURE 7


EXISTING
GRIBBS
DAM
FIRST ST
LOCATION MAP
GENERAL PLAN

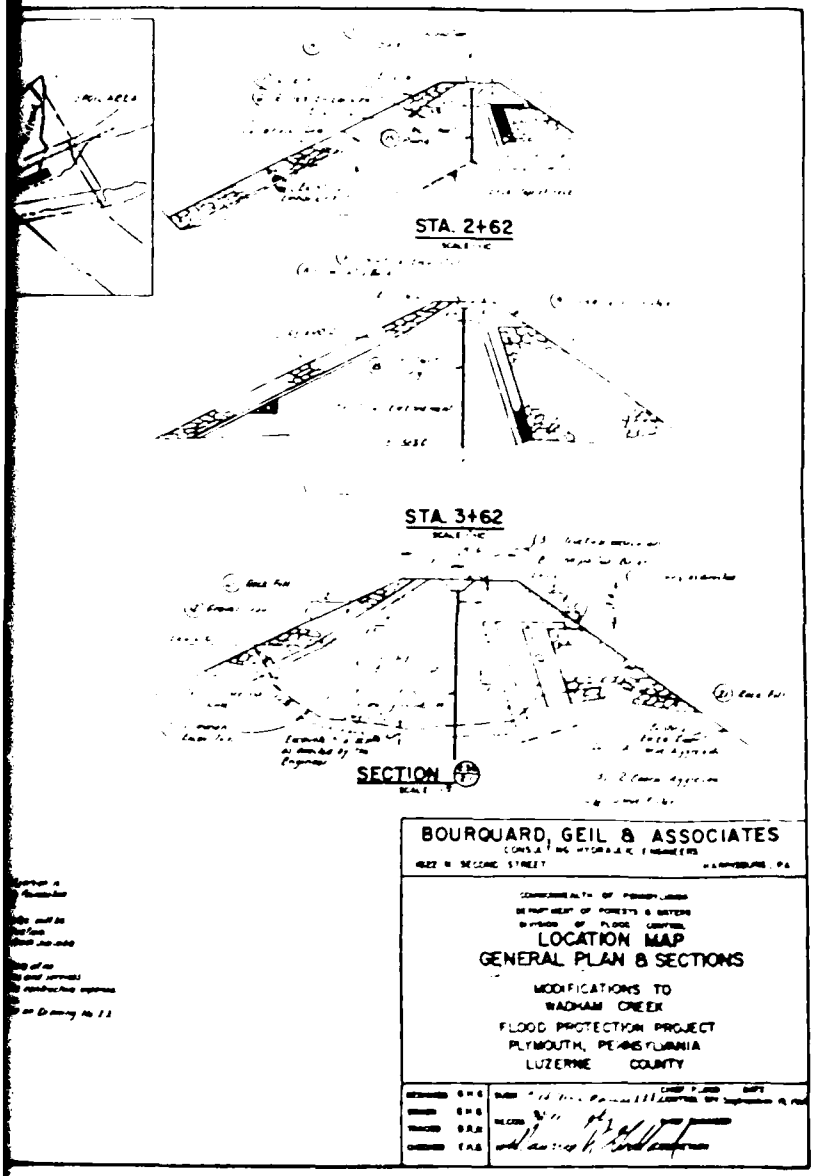


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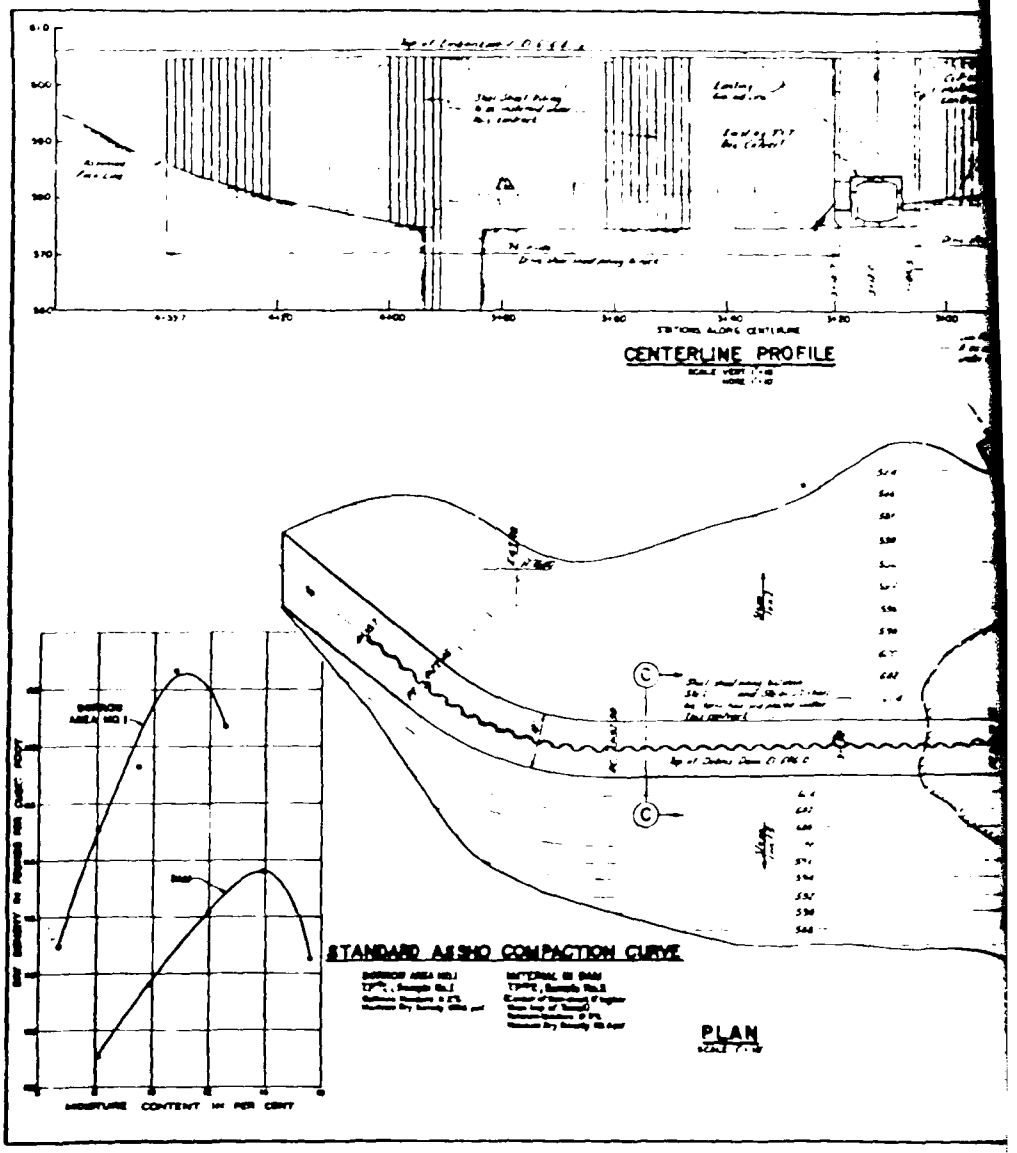
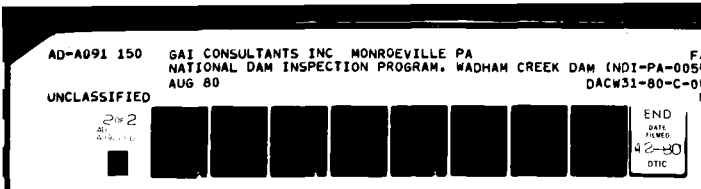
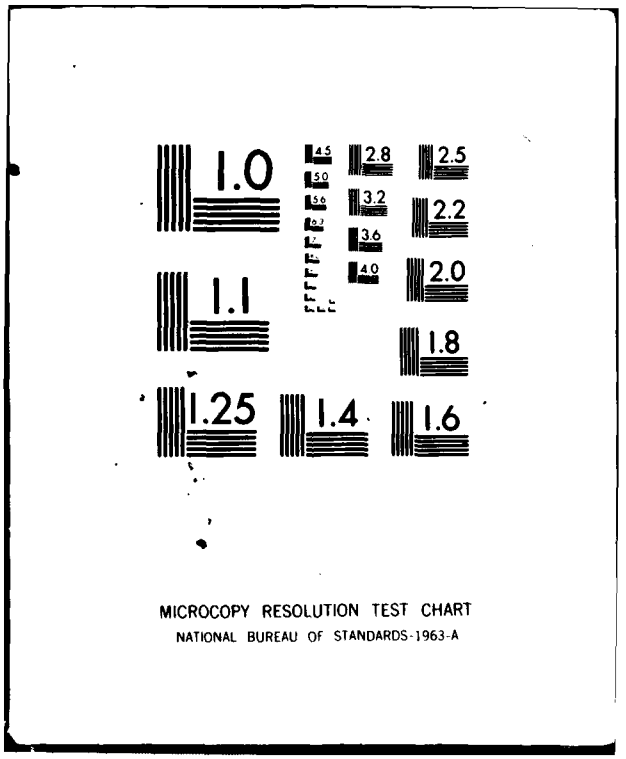
FIGURE 8


CENTERLINE PROFILE
STANDARD AASHO COMPACTION CURVE
PLAN

AD-A091 150 GAI CONSULTANTS INC MONROEVILLE PA F/G 13/13
NATIONAL DAM INSPECTION PROGRAM. WADHAM CREEK DAM (NDI-PA-00547--ETC(U)
AUG 80 DACW31-80-C-0016
UNCLASSIFIED NL




1.0
1.1
1.25
1.4
1.6
1.8
2.0
2.2
2.5
2.8
3.2
3.6
4.0
4.5
5.0
5.6
MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

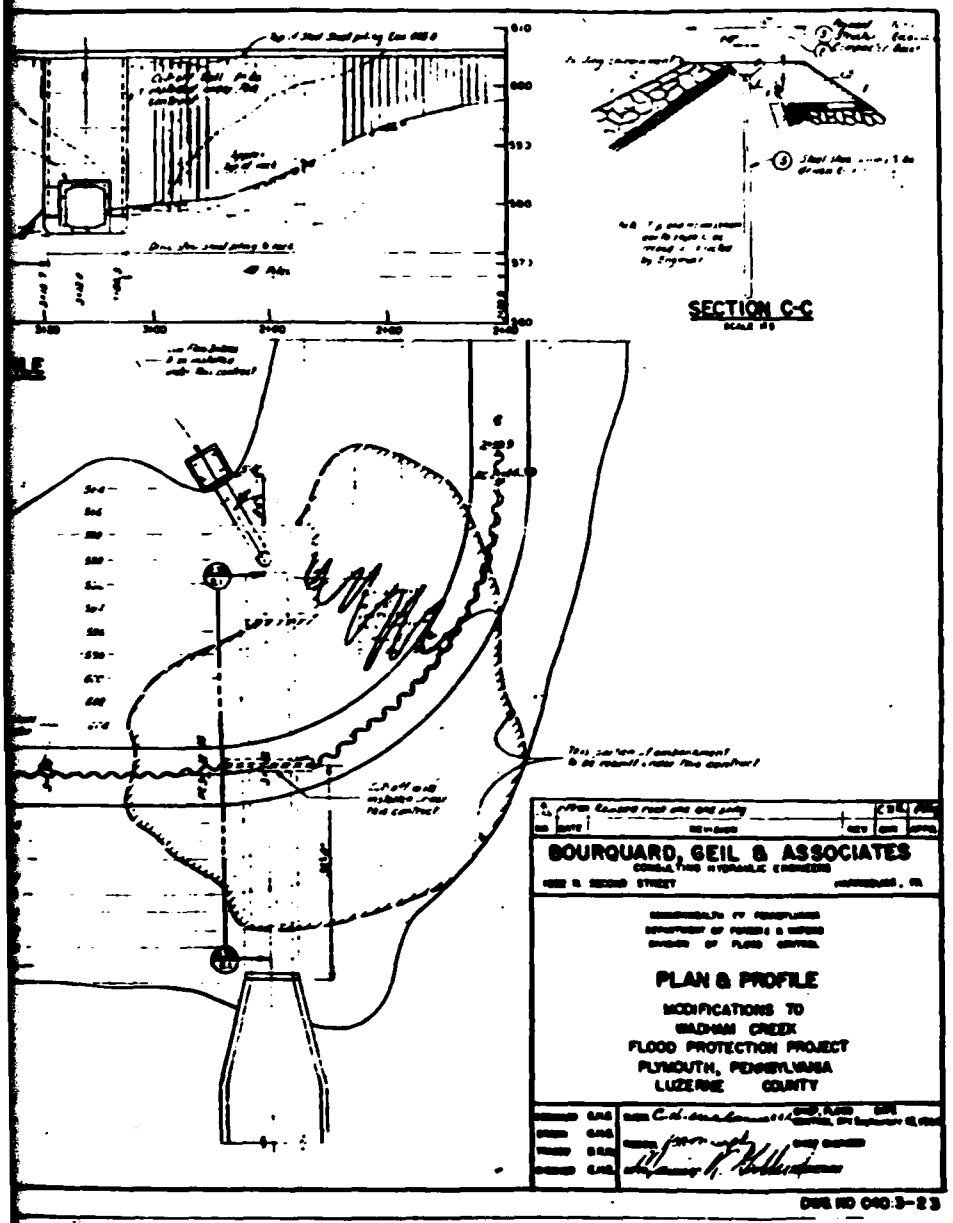


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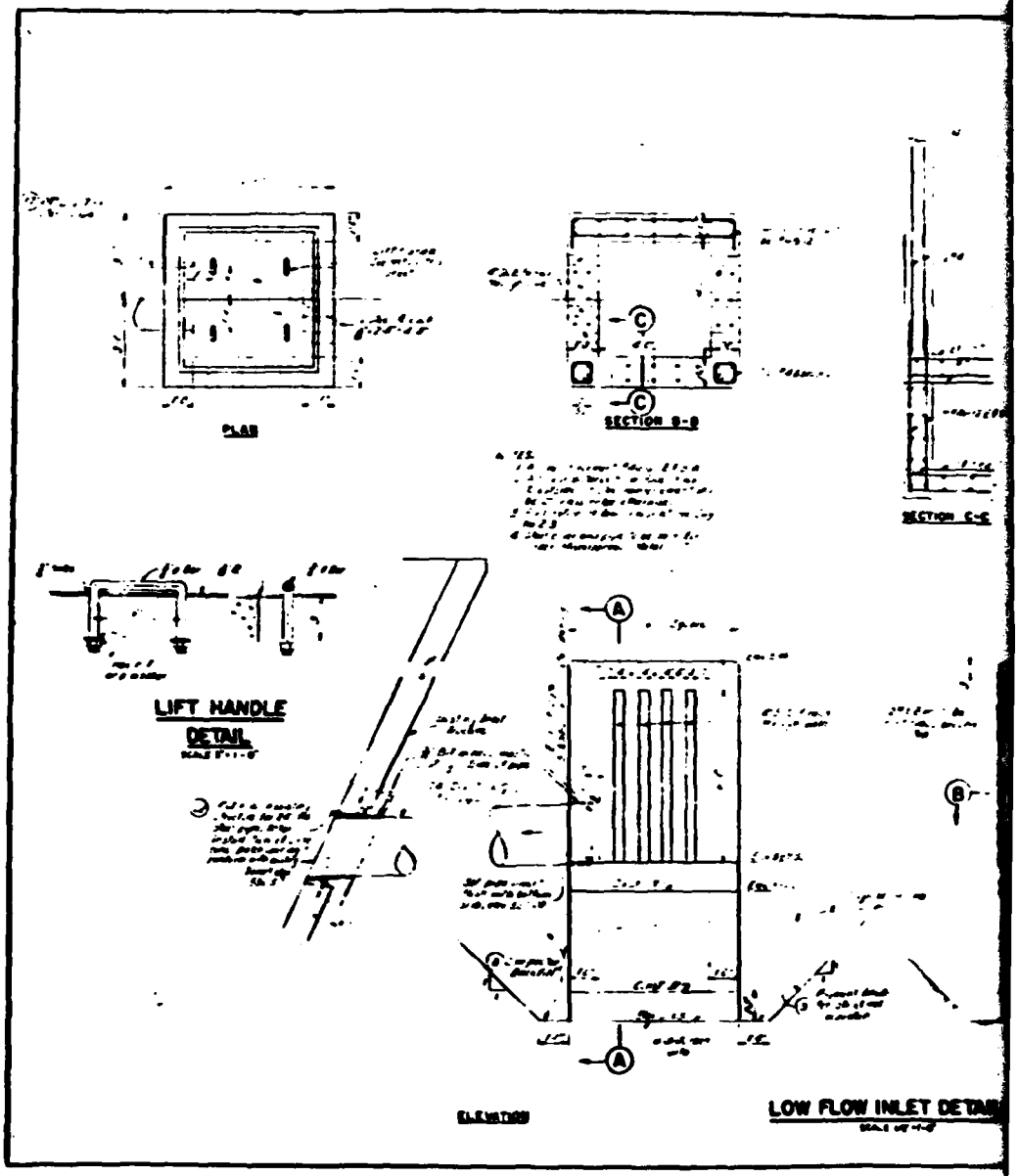
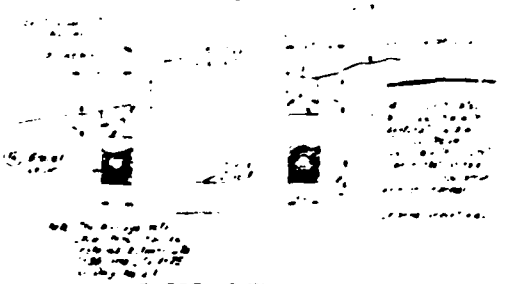
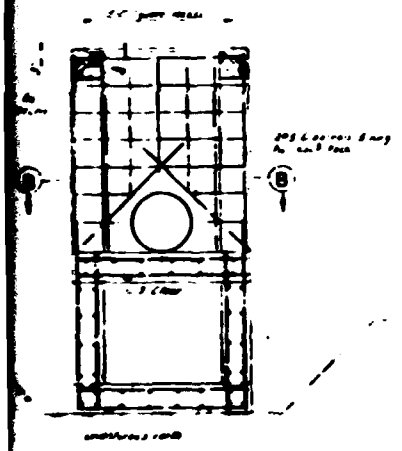
FIGURE 9


PLAN
SECTION B-B
SECTION C-C
LIFT HANDLE
DETAIL
ELEVATION
LOW FLOW INLET DETAIL



DUFFEY RUN
DRAINAGE DETAIL



SECTION A-A

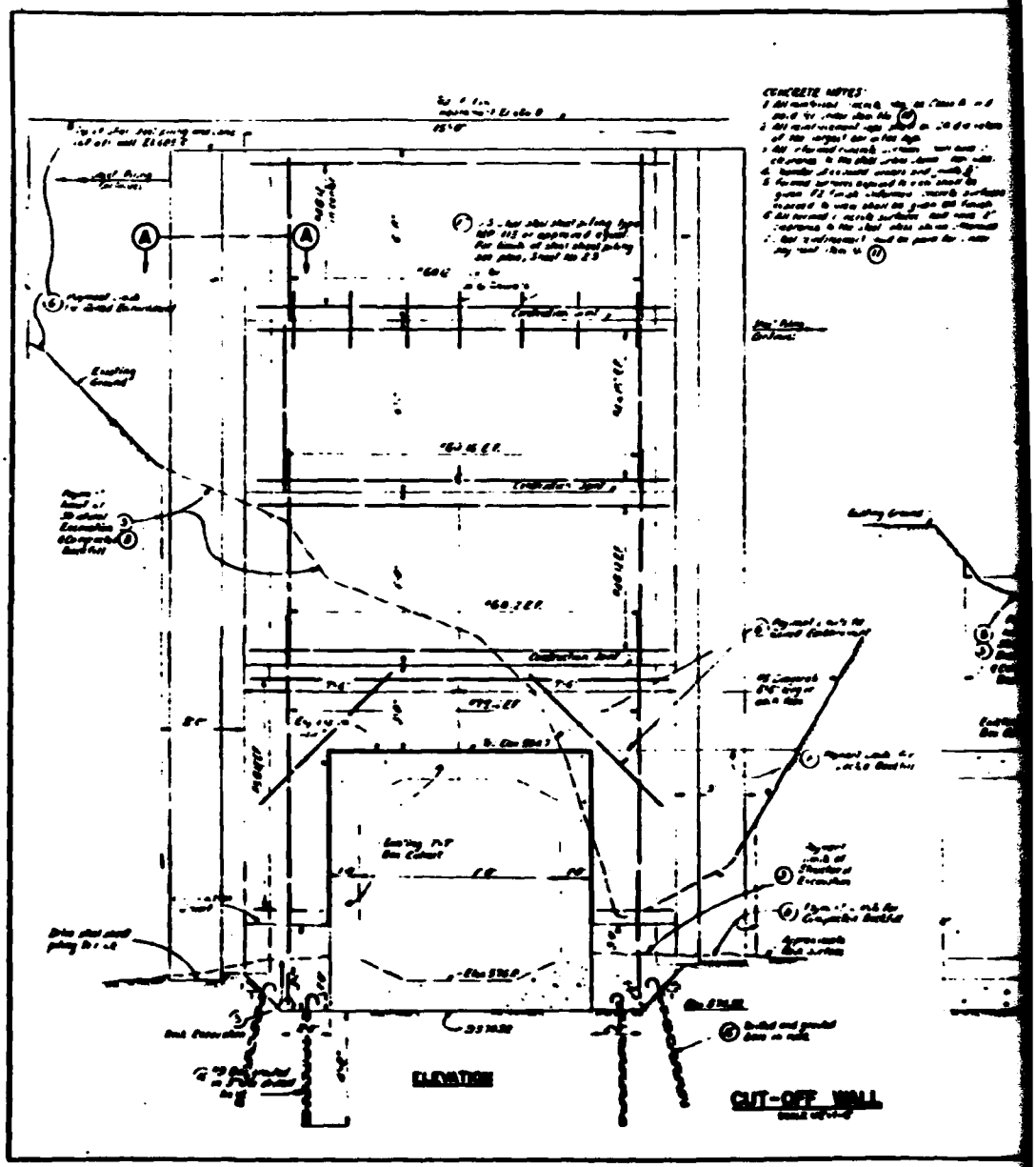
BOURQUARD, GEIL & ASSOCIATES

LOW FLOW INLET DETAILS

MODIFICATIONS TO
WADHAM CREEK
FLOOD PROTECTION PROJECT
PLYMOUTH PENNSYLVANIA
LUZERNE COUNTY

GAI CONSULTANTS, INC.

FIGURE 10


ELEVATION
CUT-OFF WALL

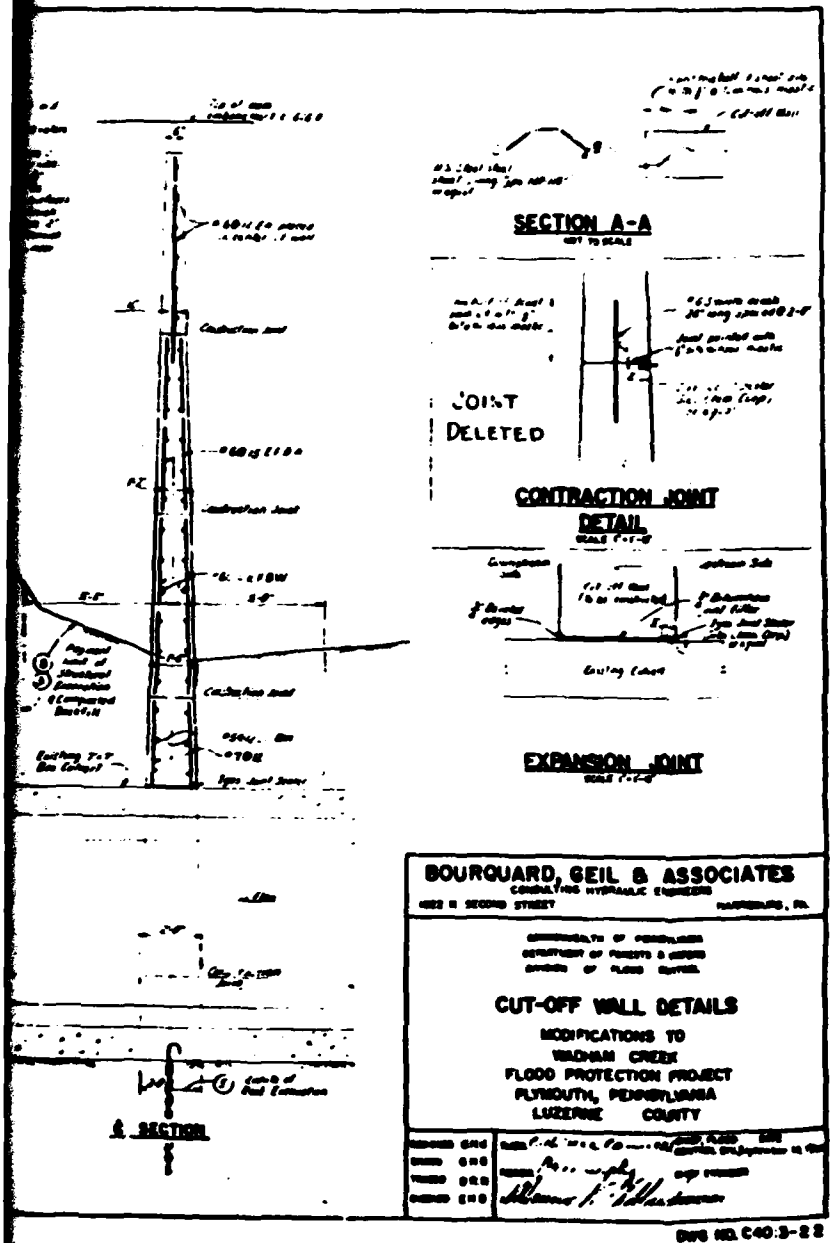


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FIGURE 11

APPENDIX F

GEOLOGY

Geology

Wadham Creek Dam is located in Luzerne County in the Appalachian Mountain section of the Valley and Ridge physiographic province, directly east of the Allegheny Front. In this area, the Appalachian Mountain section is characterized by folded sedimentary rock strata ranging from Devonian to Pennsylvanian in age. Major structural axes strike from southwest to northeast with flanking strata dipping northwest and southeast.

The dam site is situated in an area whose well developed depositional landforms were formed by glacial and glaciofluvial processes. The most conspicuous features are the Woodfoedian terminal moraine complex, which trends in a northwest-southeast direction, and the Woodfordian glaciofluvial terraces, which border the Susquehanna River.

The sedimentary rock sequence underlying the glacial tills at the dam site are probably of the Pottsville Formation of Pennsylvanian age. According to Figure 5, the thickness of the glacial till veneer overlying bedrock varies from 5 to 10 feet. The Pottsville Formation is characteristically comprised of sandstone, conglomerate, shale, fireclay, carbonaceous slate and coal, with a few beds of limestone.

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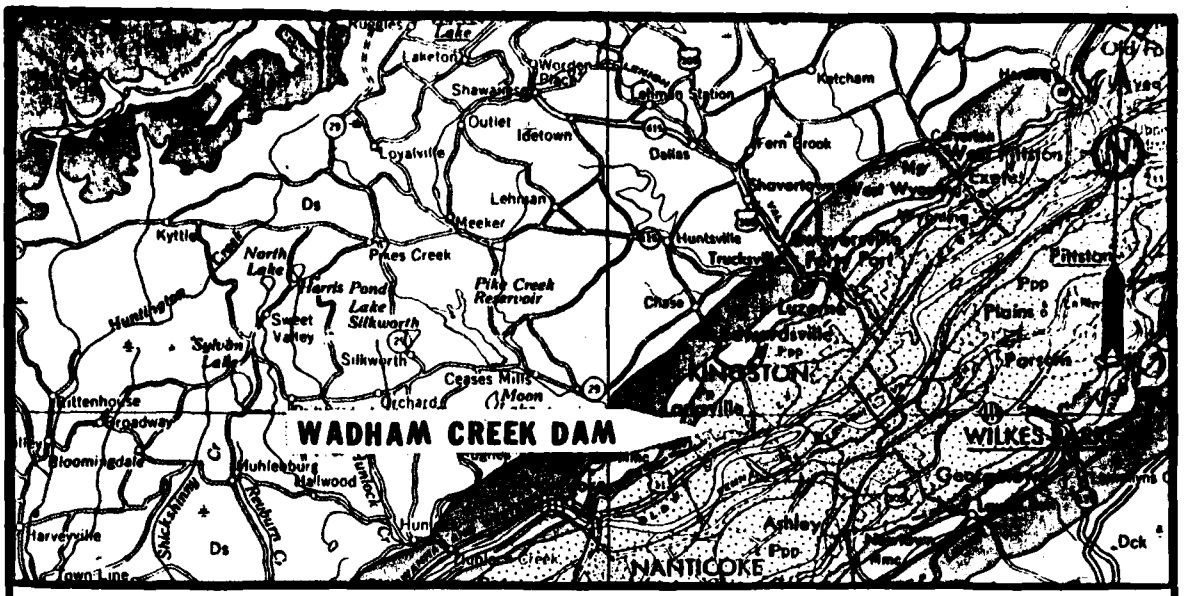

LEGEND

PENNSYLVANIAN

ANTHRACITE REGION

Ppp — **Post-Pottsville Formations**
Brown or gray sandstones and shales with some conglomerate and numerous minable coals.

Pp — **Pottsville Group**
Light gray to white, coarse grained sandstones and conglomerates with some minable coal; includes Sharp Mountain, Schuylkill, and Tumbling Run Formations.

MISSISSIPPIAN

Mmc — **Mauch Chunk Formation**
Red shales with brown to greenish gray flaggy sandstones, includes Greenbrier Limestone in Fayette, Westmoreland, and Somerset counties; Loyalhanna Limestone at the base in southwestern Pennsylvania.

Pocono Group
Predominantly gray, hard, massive, cross-bedded conglomerate and sandstone with some shale; includes in the Appalachian Plateau Burgoon, Shenango, Cuyahoga, Cussewago, Corry, and Knapp Formations; includes part of "Oswayo" of M. L. Fuller in Potter and Tioga counties.

DEVONIAN

Ds — **Susquehanna Group**

barbed line is "Chemung-Catskill" contact of Second Pennsylvania Survey County reports; barbs on "Chemung" side of line.

Note: The bedrock surface is covered with Pleistocene age Wisconsin and Illinoian till composed of sands, gravels and silty clays of variable thicknesses.

Scale

0 2 4 6 8 10 MILES

REFERENCE:
GEOLOGIC MAP OF PENNSYLVANIA PREPARED BY COMMONWEALTH OF PENNA. DEPT. OF INTERNAL AFFAIRS, DATED 1960, SCALE 1" = 4 MILES

GEOLOGY MAP

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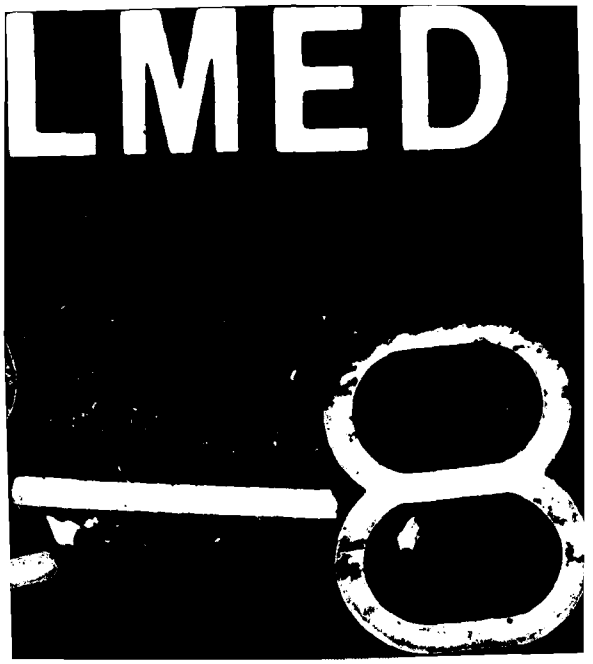
LMED